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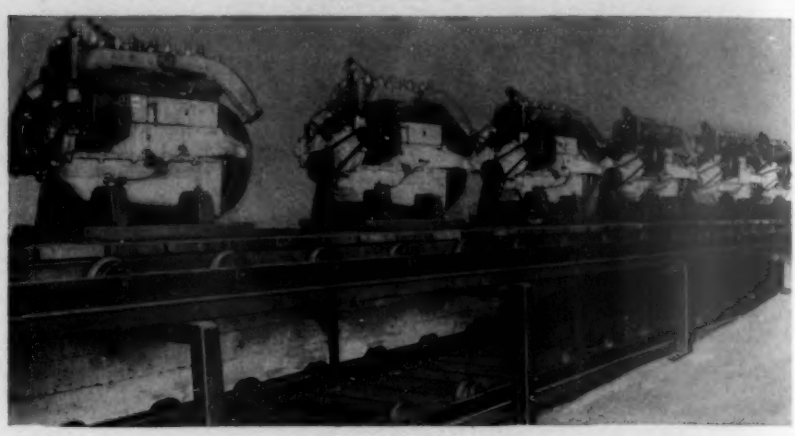

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ENGINEERING AND TRANSPORTATION.*

BY

SAMUEL REA, D.Sc., LL.D.

Retired President, Pennsylvania Railroad.

FIRST, let me thank you sincerely for the invitation to be here to-day. It is an unusual honor to be asked to deliver one of this series of lectures, and I assure you that I appreciate it keenly.

The subject which has been assigned to me is "Engineering and Transportation." Engineering is one of the professions and transportation is an indispensable form of public service. They are, however, inseparably connected. It so happens that I have been personally identified with both ever since I was given my first opportunity, on the Pennsylvania Railroad, to start my career in engineering. This was fifty-five years ago, and much has happened since then.

My interest in professional work still continues because, although I have retired from the Presidency of the Pennsylvania Railroad Company, and from active service as an officer, I am still a member of the Board of Directors and of its various committees, and scarcely a day passes in which I am not consulted on some problem of engineering intimately connected with the interests of our railroad and the general progress of transportation.

My subject for this lecture is of such vast scope that neither detailed nor technical treatment seems practicable in a paper of

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this character. To attempt to deal comprehensively with the relations between engineering and transportation would involve substantially a complete history of the development of transportation from the earliest times. Hence, I can do little more than sketch a partial outline and in addition, at Doctor Greene's suggestion, discuss for purposes of illustration certain particular engineering projects on the Pennsylvania Railroad in which I have participated directly, or have supervised, and to which I will refer later.

The wheel and axle are a basic mechanical device which appears to have been known and used from the most remote antiquity. The first wheel was doubtless the cross-section of a tree and the axle a pole made from a sapling. Their employment to convert the primitive sledge into a cart, and so facilitate movement over specially prepared surfaces—in other words crude roads or pathways—antedates the era of authentic history. The use of rails, as a specialized form of road surface, is also much older than most people suppose. For instance, there is good evidence that tramways with wooden rails, and operated by horse or man-power, were employed in the mines and quarries of Europe as early as the sixteenth century.

The use of mechanical power, however, to move vehicles over rails is a comparatively recent development. Steam railroads, both in this country and in Europe, are just completing their first century of existence. Nevertheless, that century, as we all know, has encompassed far more of progress and advancement for the human race, in a material way, than was achieved in all the preceding centuries of human experience.

SCOPE OF ENGINEERING WORK.

The profession of engineering, in its broadest scope, is the practical application of scientific progress in the utilization of natural resources, forces and laws on behalf of human advancement. To the engineer, the discoveries of pure science are not an end but a means. Technical knowledge is only his ground-work. Ultimately he is concerned with its economic and social values and adaptability.

Perhaps the best definition of civil engineering—the branch of the profession with which I have been identified—is that written more than one hundred years ago by Thomas Telford, first

President of the Institution of Civil Engineers, of Great Britain, in which I have been accorded the honor of membership since 1891. Telford declared it to be, "the art of directing the great sources of power in nature for the use and convenience of man." He further elaborated upon this definition by describing some of these uses, naming among them the means of production and traffic, the construction of roads, bridges, aqueducts, canals, docks, ports, harbors, moles, breakwaters and lighthouses, the use of artificial power in commerce, the construction and adaptation of machinery and the drainage of cities and towns. So broad a scope, of course, includes much of what we now term mechanical engineering.

I think the statement may safely be made that there is no department, phase or activity of civilized life which has not been touched and immeasurably bettered by the work of the engineering profession. I believe also the further statement may be made, with equal safety, that none of the contributions of engineering to the cause of human progress has been of greater importance than the development of transportation by steam railroads in the last hundred years. I do not make that statement because I have spent my own career almost entirely in railroad work. It reflects a wholly detached appraisal of the value of railroads, from observation of their effect upon commerce and industry and the settlement and development of the country.

SUPREMACY IN TRANSPORTATION.

The United States has grown to be the richest nation that the world has ever known. Its individual citizens have risen to, and are proving their ability to maintain, standards of living which are beyond comparison with those ever enjoyed by the citizens of any other country. This particularly applies to the great mass of wage-earners who have benefited relatively more than any other group of the population, and whose condition is immeasurably above that of persons similarly situated in any other nation of the world.

In creating the national and individual wealth of our country three main factors have been at work. The first is our natural resources, which in abundance and variety are certainly unsurpassed. The second is our economic freedom, by which I mean

the complete liberty of each individual to make the most of his talents and ability—a form of emancipation which has never been so fully realized by any other people. The third is our unmatched systems of transportation. By this I mean primarily our steam railroads, which are, and will continue to be, our fundamental means of transportation, and, secondarily, the highways with our automobiles and other motor vehicles.

When we consider that we have about two-fifths of all the railroad plant in the world within our boundaries and about four-fifths of all the world's motor cars, that our per capita use of rail transportation is from seven to fifteen times that of the most developed countries of Europe, and that the entire population of the United States could be loaded into our motor vehicles simultaneously, we begin to gain some faint idea of the vast extent to which the American people have employed transportation, and how greatly they have surpassed the rest of mankind in this respect.

It has been said that the function of transportation is to overcome the economic handicaps of distance. Because we have created in this country the most extensive systems of railroads, and the lowest cost and most efficient transportation, we have been able to develop and utilize our natural resources and labor power to better advantage than any other country in the world.

From the viewpoint of an iron or steel maker, the distance of the Minnesota ore mines from Pittsburgh is not measured in miles but in the cost of getting a ton of ore from the mine to the mouth of the furnace. The efficiency of our railroad transportation has therefore enabled the iron and steel industries to avail themselves of the great advantages which such centres as Pittsburgh present with respect to other materials, labor supply and the location of markets. It has made possible the economies of centralized manufacturing as typified, for instance, in the automobile plants of Detroit. In short, efficient transportation by rail not only led to the original opening up and settlement of our country, from boundary to boundary, but has since furnished the basis upon which our present-day system of industrial mass production has been shaped. And upon mass production in industry the real commercial supremacy of America rests and must continue to be sustained.

RAIL PROGRESS FROM MECHANICAL SIDE.

From the viewpoint of physical plant, the progress in transportation during our hundred years of American railroad development constitutes an outstanding tribute to the vision and genius of the engineering profession. In this honor, the various branches of engineering have had their respective shares.

The mechanical advance may be visualized by a comparison between the oldest and the newest locomotives used on what is now the Pennsylvania Railroad System. The earliest of these engines, the "John Bull," now preserved in the Smithsonian Institute at Washington, was first operated in 1831 on what was then the Camden and Amboy Railroad and is now a part of our Trenton Division. It weighed eleven tons and in practical use probably developed about the power of an ordinary automobile of the present day. The standard heavy duty freight locomotive now used on the Pennsylvania Railroad, our "I-l-s type," weighs nearly two hundred tons without the tender, and at its most efficient speed develops four thousand horse-power. Thus it is certainly conservative to say that in ninety-five years we have multiplied the effectiveness of our mechanical unit by at least one hundred.

PART PLAYED BY CIVIL ENGINEERING.

In the field of civil engineering, with which, as I stated a moment ago, my own career has been identified, the comparisons are somewhat more complicated. The earliest lines, including all of what are now the large eastern and southern systems, were projected as local enterprises to serve local needs. The Pennsylvania, for instance, was chartered to build a line from Harrisburg to Pittsburgh and in that manner, by utilizing the already existing Philadelphia and Columbia Railroad, link the head of navigation on the Ohio River with tidewater at Philadelphia. Its purpose was to protect the commercial interests of Philadelphia against the competition of New York, on the north, and Baltimore, on the south, for the growing trade then springing up with the new West.

Some years elapsed after the granting of the charter and initial construction of the Pennsylvania Railroad before it began to take shape as a trunk line system, and the conception of rendering a national and not merely local service arose. But about the time of the completion of its through line to Pittsburgh, the management began to reach out toward Chicago and St. Louis, by

investments in lines then being promoted, many of which were subsequently leased and are part of the present system. However, it was not until the era of the projection and construction of the Pacific roads, in the Civil War period and later, that American railroad builders progressed to the deliberate planning anew of trunk lines or trans-continental systems, as such.

The first rail lines were for the most part projected and built as single track and, for the sake of cheapness in construction, closely followed the natural contours of the country, thus permitting a minimum of cuts, fills, bridges, tunnels and other forms of expensive construction necessary when we depart from natural contours. Traffic grew very rapidly and in many cases early necessitated the building of second track. In most instances this was done with the greatest possible economy and speed and with very little revision of line.

The result was that later, when third and fourth, or even more running tracks became necessary, and heavier loads required elimination of curves and lower gradients, extensive and in many cases practically entire reconstruction of the line was required. An illustration of this is to be found in the present Main Line of the Pennsylvania Railroad between Philadelphia and Harrisburg. The distance is one hundred and three miles and out of that only a little more than twenty miles of our main track are upon the original location.

Perhaps I can best picture the advancement which civil engineering has contributed to American railroads by continuing with our own road as an example. The Pennsylvania Railroad was chartered March 31, 1846, only eighty years ago and not a very long time in human history. At that period, passengers and freight moved between Philadelphia and Pittsburgh over one of the most remarkable transportation systems the world has ever seen. Its first link was a primitive rail line between Philadelphia and Columbia, Penna., operated, at the start, with horses as motive power. Next came a stretch of canal up the Susquehanna River from Columbia, through Harrisburg, to the mouth of the Juniata; thence along the Juniata Valley to Hollidaysburg, at the eastern foot of the Alleghany Mountains. The third link was the Allegheny Portage Railroad, built across the mountains to their western foot at Johnstown. It consisted of a series of eleven inclined planes connected with levels, upon which the canal boats, built in

sections and taken from the canal and mounted on wheeled trucks, were hauled up and lowered over the slopes of the Alleghanies. The inclined planes were worked with stationary engines and cables. Horses were at first used on the level stretches, and later steam locomotives. The fourth and last link was another stretch of canal from Johnstown to the head of navigation on the Ohio River at Pittsburgh.

This combined system of canals, rail lines and inclined planes was known as the " Main Line of Public Works " and was owned by the State of Pennsylvania. It was operated by the State Government for about twenty years, beginning with 1834. During that time it developed, in most aggravated form, all of the customary and inevitable evils of government ownership and management, from waste and wretched service to flagrant and widespread corruption and dishonesty on every hand. It finally passed from existence, leaving behind an indebtedness of $50,000,000, a stupendous sum for that day.

The journey from Philadelphia to Pittsburgh took anywhere from three days to a week, and its discomforts and difficulties have been graphically described by Charles Dickens in one of the most memorable chapters of his " American Notes." To cap the disadvantages, the canal divisions were of course closed by ice several months of the year—a defect, by the way, necessarily inherent in canal or canalized river transportation anywhere in Canada or the United States, except the far South. A complete account of this transportation system may be found in a historical compilation entitled " The State Works of Pennsylvania," by A. L. Bishop, Ph.D., and published by Yale University Press.

Parenthetically, let me take this occasion to express my opinion that, in our country at least, inland canals are as obsolete and inadequate to present-day needs as stage coaches or Conestoga wagons, and we might as well go back to the one as the other. Those who are advocating artificial waterways throughout the country seem to be doing so on the theory that the railroads are or will be congested and will require the canals for relief. This appears to involve the further assumption that the railroads are henceforth going to stand still and will not continue, as in the past, to meet the growth of traffic, and the needs of patrons, by increasing their trackage and other facilities, and through advantageous and economic consolidations.

Such theories and assumptions are wholly fallacious and should certainly be completely discredited by any fair consideration of the history of railroad transportation in this country and the place of leadership which it has always occupied in our national progress.

The truth is that the high development of the railroads, their powerful locomotives and big freight cars, carrying coal and other minerals from point of origin to point of consumption regardless of climatic and other physical obstacles, together with the continuous fall in freight rates down to about 1900, made it impossible for the canals to maintain successfully any real competition. Costs of canal construction and operation, and the limited scope of industries and communities which a canal can serve without rehandling of the barge loads, still keep railroads as the cheapest method of transportation. A study of statistics will show that since about 1880, canal projects have been successively abandoned in this country, or, in those cases where they have been continued, their operation has been conducted at a loss.

Returning to the " Main Line of Public Works " of Pennsylvania, as existing eighty years ago, and to the cumbersome, inadequate and easily interrupted service then afforded between Philadelphia and Pittsburgh, together with the total discontinuance of service throughout the winter, I will ask you by way of contrast to consider the present main line of the Pennsylvania Railroad between these two cities. Bear in mind that it has not less than four tracks of the heaviest rail all the way, and in many places six to eight tracks, visualize the immense stream of traffic which it carries day and night in heavy passenger and freight trains, and further compare their speed and dependability of movement, rarely interrupted even for an hour, with the performance of the canal boats, and you will complete a brief, but I believe quite illuminating, survey of the progress which civil engineering has brought about in the advancement of rail transportation.

WORK OF ELECTRICAL AND CHEMICAL ENGINEERS.

Electrical engineers entered the railroad field at a somewhat later date than the civil and mechanical, but their work has been extremely important. It is notably represented in highly developed form to-day by our automatic block signals and our systems

of telephone and telegraphic train dispatching. Every railroad executive also realizes the value and possibilities of electric motive power. The fact that out of 260,000 miles of railroad in this country less than 2000 miles have thus far been electrified, does not reflect lack of progressive spirit nor infrequency of dense traffic conditions warranting electrification. It reflects rather the cramping influence of an inadequate return on property investment during a number of years in the past. This, in turn, is a reflection of harsh policies of governmental regulation, now happily being liberalized in response to the more constructive spirit of our day.

Chemical engineering, too, has its place in railroad progress. I cannot attempt to elaborate upon it in detail, but will merely point out that it embraces, for example, the production of new alloys and the perfection of heat treatment for axles and other parts where unusual strength is needed—processes, incidentally, which will probably prove applicable in the further improvement of steel rails. From such problems as these, the province of the chemical engineer in railroad work ranges to the working out of improved paints and other protective surfaces for the preservation of cars, bridges, etc., from the effect of weather and corrosion; and with the movement toward universal adoption of steel equipment, this will certainly prove of increasing consequence.

That all branches of the engineering profession will continue their contributions to railroad progress, and that these contributions will be of even greater importance in the future than in the past, I have not the slightest doubt. The problems of the future, indeed those now already arising, promise to be more interesting than ever before.

The growth of our national needs for transportation seems to be unlimited and the situation is rendered more complicated, but also more worthy of our best efforts, by reason of the fact that henceforth railroad progress cannot be considered alone. It must be considered in connection with coördinating rail transportation with motor and air transport. These are subjects which I should like to discuss at some length, but the limitations of time forbid. I can only state my views, which I have already indicated, that the real problem, as between railroads, motor cars and airships, is one of coöperation rather than competition. Each, I think, will have its own field in which it will be distinctly superior;

further, by proper coördination of these various agencies of transportation much additional traffic will be created, and in that way alone can each render its greatest service to the public.

TUNNEL PROJECTS RESULT OF LONG STUDY.

When I accepted the invitation to come here, Doctor Greene, doubtless realizing the unlimited scope of the subject which has been assigned to me, and the impossibility of a complete survey, suggested that the treatment of some concrete examples of engineering works would be of interest. He indicated for this purpose the construction of the Pennsylvania Tunnel and Terminal properties in the vicinity of New York City, which, together with the Hell Gate Bridge and New York Connecting Railroad, really constitute a single project of engineering. I am happy to accept this suggestion, particularly as I was continuously associated with the planning and consummation of these works from the first serious studies of the development of underground railroad construction in London, which I made in 1887 and 1892, to the completion and opening of the Hell Gate Bridge in 1917.

First, let me disabuse your minds of the popular error that the determination to tunnel New York City and the two great rivers which bound it was a sudden inspiration or off-hand decision of the late Alexander J. Cassatt, who filled the Presidency of the Pennsylvania Railroad Company with such conspicuous ability from 1899 to 1906. I doubt whether any important executive ever lived who was less given to snap judgments or spectacular actions than Mr. Cassatt. He never entered upon a weighty undertaking in his entire career without complete mastery of the facts, and of all the important projects with which he was identified, none received more painstaking care, investigation, thought and study than were expended, over so many years, upon the problems incidental to the extension of our railroad system into and through New York City.

Let me say, however, that Mr. Cassatt was perfectly capable of reaching important conclusions quickly and brilliantly, and did so daily. Otherwise, he would not have proved the remarkable executive we know him to have been. But such decisions were always founded on certain knowledge and long experience and reflected the operations of a splendid memory, a cultivated mind and a faculty, which amounted to genius, of combining

similar experiences and concentrating their lessons immediately upon the subject before him. He never guessed and he never ventured blindly into the unknown.

NECESSITY FOR PLAN EARLY REALIZED.

As a matter of fact, realization of the desirability, or indeed ultimate necessity, of obtaining a direct entrance for the Pennsylvania Railroad into New York City antedated Mr. Cassatt's time as President. It must have been obvious to our directors as early as the consummation, in 1871, of the lease of the United Railroads of New Jersey, by which the Pennsylvania Railroad lines were brought to tidewater at New York Harbor on the Jersey City side. As long as we remained on the west bank of the Hudson River we were at an unquestionable disadvantage with our chief rival, the New York Central, which entered the city from the north, *via* Albany, and hence had never been faced with the difficulties of construction which confronted us. That we were able for years to maintain a competitive footing against these undeniable odds is a fine tribute to the enterprise and ability of the Pennsylvania management of that day.

In 1874, the earliest actual attempt at tunnelling the Hudson River was started by DeWitt C. Haskins. It was stopped by injunction and not resumed until 1879. In 1880, a blow-out occurred in the New Jersey shaft, resulting in the loss of twenty lives, which stopped all talk of a North River tunnel for many years. This fact, I presume, has passed from the memory of most people of to-day.

When these events occurred, the late Thomas A. Scott was President of the Pennsylvania Railroad and George B. Roberts, Vice-president, both being predecessors of Mr. Cassatt as chief executive. The management, we know, had been approached and exhibited interest in the tunnel scheme, but our officers did not approve the use of steam locomotives in sub-aqueous tunnels and electric traction had not been developed.

In 1884, when I was Assistant to Vice-president J. N. DuBarry, Mr. Gustav Lindenthal, Civil Engineer, of Pittsburgh, prepared a plan for extending the Pennsylvania Railroad into New York by a high level bridge over the Hudson River. This also interested Mr. Roberts, as well as other of our officers, including Mr. Cassatt, as a Director of the Pennsylvania. Defi-

nite plans for such a bridge were actually worked out under a Federal charter, which was granted to the North River Bridge Company in 1890; but the project was not realized, due to the fact that we could not obtain the coöperation of the other railroads, and it was impracticable for the Pennsylvania Railroad alone to bear the burden.

In 1892, I made a study of the London Underground Tubes, as then building and proposed, and upon my return presented a report to President Roberts, which dealt especially with a project for extending a rapid transit tube railway from Jersey City to the Long Island Railroad in Brooklyn. This was promoted by the late Austin Corbin, President of the Long Island, and officers of the Pennsylvania Railroad, while Charles M. Jacobs, of London, and myself, were the engineers. The panic of 1893 and the ensuing six years of business depression stopped this work and indeed active consideration of all extension plans. Part of this route was, however, later covered by the Hudson and Manhattan Railroad, in coöperation with the Pennsylvania. The bridge project, plans for which had been approved by the Secretary of War, was then resumed, but with the result, or rather lack of result, which I have just stated. In 1900, the Pennsylvania Railroad had bought a controlling interest in the Long Island Railroad with the ultimate object of constructing a railroad through New York City to connect with the New England railroads.

In the summer of 1901, Mr. Cassatt, who in the meantime had become President of the Pennsylvania Railroad, was in Europe. I cabled to him suggesting that he inspect the Extension of the Orleans Railway in Paris, then nearing completion. It was my thought that the advance in electric traction, which this steam railroad extension exemplified, might make a tunnel under the Hudson River possible. Mr. Cassatt was favorably impressed with the Extension and the possibilities of electric operation, and upon his return set actively to work upon our own New York tunnel and terminal project which from that time was pushed steadily to conclusion.

Unfortunately, and to the great regret of all of us, Mr. Cassatt did not live to see the work more than half done. His name, however, is inseparably linked with the planning and

execution of these great enterprises which constitute a noble memorial to his vision, capacity and unsurpassed talent as a railroad builder and executive.

LEGISLATION AND ORGANIZATION REQUIRED.

For such a great undertaking, involving so large an expenditure of capital and requiring five or more years for accomplishment, it was necessary, as preliminary steps, to create a special engineering organization as well as insure that the enterprise should in all respects be properly, legally and permanently founded.

We found that the existing laws permitted only a limited franchise and contemplated railroads beginning and ending on the surface, and in places of human habitation. An extension of the character which we contemplated, however, could not prudently and properly be entered upon without a perpetual right to build and operate, though the annual franchise payments might be subject to revision at fixed periods. Moreover, the project, as far as the New Jersey franchise was concerned, involved a termination about eighty feet below high water in the middle of the Hudson River on the New York-New Jersey State line, at which point, by a tunnel, it would connect with a similar railroad built and owned by a New York corporation.

Therefore, it was necessary to secure amendments to the law. It is a pleasure to say that the legislators and others, appreciating the importance and public value of the work, facilitated revision to the fullest extent. Nevertheless, the action of two legislatures was required before we could accept the law and go forward with the acquisition of property and other work preparatory to construction.

The form of organization which we decided upon for conducting the work of construction, and for professional advice and supervision, was a Board of Engineers, composed of the several Chief Engineers and others not directly connected with the construction. The function of the Board was to examine and approve all plans in detail, and recommend specifically to the management.

While President Cassatt, as chief executive of the company, was of course primarily responsible to the Directors and stockholders for the successful carrying out of the project, the Board delegated to myself, as Vice-president, subject to Mr. Cassatt's oversight, the active direction and personal supervision of the

organization and construction work, including the acquisition of property required. Although both Mr. Cassatt and myself were civil engineers, it was decided that it would be undesirable for either of us to be members of the Board of Engineers, because on us devolved the final decision for the management. Accordingly, the latter as finally organized consisted of the following gentlemen:

William H. Brown, Chief Engineer of the Pennsylvania Railroad, who was given charge of construction from connections with the New York Division at Manhattan Transfer, near Newark, to the west end of the Bergen Hill tunnels;

Charles M. Jacobs, who was appointed Chief Engineer for the Bergen Hill and North River tunnels;

Alfred Noble, who was designated Chief Engineer for the tunnels under the Boroughs of Manhattan and Queens, the excavation of Pennsylvania Station site and Sunnyside Yard, Long Island;

George Gibbs, designated Electrical Engineer and placed in charge of Pennsylvania Station construction;

Gustav Lindenthal, Consulting Engineer;

Colonel Charles W. Raymond, U. S. A. (later retired as Brigadier-General), Chairman of the Board.

Messrs. McKim, Mead and White were the architects for the station.

DUTIES AND PROBLEMS OF THE BOARD.

The first duty of the Board was to prepare the plans, which required much work and time. One of the problems had to do with the probable behavior of sub-aqueous railroad tunnels constructed through material such as that forming the bed of the Hudson River. Test borings showed rock at both sides of the river, but at its centre a silt of varying character and consistency and 180 feet in depth. It contained considerable water and was capable, at times, of flowing under pressure. As the proposed tunnels would weigh less than the material which they displaced, it was feared by some engineers that they would have a tendency to float or rise upward during and after construction. However, experiments which led to the determination of elaborate formulæ based upon the size and weight of the tunnels and their "angle of repose," disposed of this fear and actual experience showed

that the tendency of the tunnels, both under construction and afterwards, was, generally speaking, to subside or settle rather than to float upward, particularly during disturbance of the surrounding silt.

Other engineers believed there was a drift, or tendency of the silt to move southward with the flow of the river and that this drift might shift any tunnels built through it unless they were anchored in some extremely secure manner. Tests and observation disposed of this fear also.

Another question, and one which proved more difficult of determination, arose in the early stages of the project. It was whether, in view of the tendency of the tunnels to settle rather than rise, they should be supported through the silt by screw piles or otherwise. The prevailing opinion of the Board at first was that supports of this kind should be used and the preliminary plans provided for them. It must be remembered that our tunnels were the first built for heavy railroad traffic through the Hudson River bed or any other similar silt, and we were, therefore, going into an unknown field of engineering work.

While the majority of the Board at first favored using such supports, they were able to give the management no indication of the measure of insurance that would be gained by the introduction of this expensive and largely experimental feature. Nevertheless, we made provision in the design of the iron tubes for the introduction of such supports every fifteen feet and circular plugs were left at the bottom of the lining at these intervals, so that in case we should determine to put in the supports the work could be carried out.

As the work progressed, we made a series of experiments with screw piles and plain cylinders by sinking them from the interior of the tunnel down through the silt. This caused a great deal of disturbance of the silt, and more or less settlement usually followed.

Opinion in the Board continued to grow more divided as the work proceeded. Finally the time arrived when a decision had to be made as to whether we would or would not use the supports. Meanwhile, experiment and observation revealed the fact that there was a slight daily vertical movement of the tunnel tubes in the silt, corresponding to the rise and fall in the tide, and for that reason called the "tidal oscillation." It averaged, and still

averages, about one-eighth of an inch and therefore, in the event supports were to be placed under the tunnels, the question immediately arose as to whether the tubes should be fastened to them or whether some sort of sliding device would have to be adopted which would permit of this "tidal oscillation" without injury either to the shell or the support.

As we were unable to obtain a definite recommendation from the Board of Engineers, owing to the division of opinion which prevailed, and as the work had to be completed, it was necessary for someone to take the responsibility for making the decision so that the project could be carried out. This responsibility devolved upon myself.

After the most careful consideration, I became satisfied that the supports were unnecessary, and that they even might lead to positive injury to the tunnels. I therefore decided that we would complete the tunnels without any additional support, and proceed with the reinforced concrete lining throughout; and the work was carried out in that manner. The openings for the supports were sealed but still remain there in case the future should develop necessity for their use.

NO IMPORTANT TUNNEL MOVEMENTS OBSERVED.

Up to the present time, after fifteen years of use, nothing indicates that such necessity is ever likely to arise. We carry on constant observations of the very slight movements which take place in the tunnels. Besides the daily oscillation of about one-eighth of an inch, the following changes have occurred in the fifteen years since the tunnels were opened: In the first year after construction, but before operation started, there was an average subsidence of about four inches on the New Jersey side, eight inches on the New York side and five inches under the centre of the channel. Following that, under conditions of actual operation, a further subsidence took place, averaging 1¼ inches on the New Jersey side, 2⅜ inches on the New York side, and ⅝ of an inch under the channel. More recently channel dredging in various places, to accommodate the very deep draft ships now using the river, has caused a slight rise in portions of the tunnels running from ⅜ of an inch to one inch. None of these variations, however, affected the stability of the tunnels.

I may add that the sealing and lining of the tunnels, by means

of reinforced concrete, have been so effective that the daily seepage is only about 400 gallons in two miles of length. This, in connection with the extremely small movements which have taken place in so large a piece of construction, appears remarkably satisfactory from the engineering point of view, particularly in view of the fact to which I have already referred, that the construction of such tunnels for heavy rail traffic, through such material as the Hudson River silt, presented entirely new problems to our profession.

The building of the tunnels was carried out by the shield method of progression, with which you are doubtless generally familiar. The tunnels were pushed forward from both sides of the river until they met, with remarkable accuracy, in the centre. The tubes themselves consist of cast-iron cylinders, 23 feet in external diameter and composed of segments 1½ to 2 inches thick bolted together by means of heavy flanges. The iron tunnel cylinders alone are amply sufficient to carry the traffic and resist the pressure, but, as a further precaution, we deemed it wise to add the reinforced concrete lining.

PENNSYLVANIA STATION AND ITS PROBLEMS.

I can refer only briefly to the Pennsylvania Station itself. You may be interested in the fact that it covers more territory than any other building ever constructed at one time in the history of the world. The Vatican, the Tuileries and the Winter Palace at Petrograd are larger buildings, but were centuries in construction. The Pennsylvania Station is unique in the fact that, covering eight acres of ground, with exterior walls approximately a half-mile in length all told, it was erected in less than six years' time.

Twenty-five years ago the intersection of 6th Avenue and Broadway at 33rd Street looked inviting as the site for our station. It compared with Charing Cross in London and other central city stations. Study and reflection, however, upon the development which would follow the extension of the Pennsylvania into and through New York, plus the natural growth of traffic in the city, led us to the belief that the choice of that location would be a mistake and surely lead to undesirable congestion. We therefore decided to drop back a block to 7th Avenue and provide access from all sides of the station with connec-

tions through to 34th Street. This was done and time has fully justified our choice. Now, forty million passengers are using the station every year.

We at first contemplated and fully intended utilizing the air rights of the station, and a large hotel and office building were designed as overhead structures. When we considered, however, that our main object was to build a railroad station which had to be located over tracks and platforms forty feet below the street level, we felt that the real use would be so subordinated, and the available space at platform level so contracted, as to reduce materially its capacity and facilities. We therefore discarded these plans and designed and built a railroad station with such office space as could properly be adapted without minimizing its use as a station, while at the same time also utilizing the surrounding areas to the best advantage.

An important section of the surrounding area, as you all know, we utilized in building the Hotel Pennsylvania and leasing it to the Statler Hotel organization. The hotel is erected partly over our tunnels which run from the station, under 32nd and 33rd Streets, across New York City to Long Island. Other properties likewise acquired in the neighborhood of the station will also gradually be developed.

HELL GATE BRIDGE AND CONNECTING RAILROAD.

Our entire system of New York improvements was carried to its logical completion by the building of the Hell Gate Bridge and the New York Connecting Railroad. This project, in conjunction with the Hudson River and East River tubes and the tunnels under New York City, links the Pennsylvania Railroad directly with the New Haven System and thereby provides New England with a direct all-rail passenger outlet, *via* the Pennsylvania Station, to the Middle States, the West and the South. Freight is exchanged between the two systems, *via* the New York Connecting Railroad, the Hell Gate Bridge and the Bay Ridge terminals of the Long Island Railroad, thence across upper New York Bay, by carfloats, to the tracks of the Pennsylvania Railroad System at Greenville Yards, N. J.

The Hell Gate Bridge, an extraordinary structure and one of the most majestic and imposing ever erected, spans the East River with a thousand-foot steel arch, the largest in the world,

carrying four railroad tracks to meet the highest requirements of traffic. Its beauty, its striking appearance, and its utility are a tribute to the genius of its Chief Engineer and Architect, Gustav Lindenthal, who designed and carried out its construction. A complete history, with all details, of this important structure can be found among the proceedings of the American Society of Civil Engineers.

From the local viewpoint, it was of course most important that our entire tunnel and terminal project in New York City should be closely correlated with city transportation facilities and particularly with the subways. It was our expectation that the 7th Avenue subway would be completed at the time the station was opened. Though we were disappointed in this and it was not ready for traffic for some years later, all required facilities have since been completed, and through an underground passageway, it is possible to go from our station to any point reached by the connecting New York subway lines without going above ground. In like manner similar connection will be effected with the new 8th Avenue four-track subway.

I fear that I have now taken up as much of your time as I can rightly claim and while I have touched only a few of the higher lights, will not detain you longer. I thank you again for the honor of your invitation, for the pleasure and privilege of being here, and for your courteous attention.

Uses and Handling of Liquid Chlorine.—This element in liquid form has become a commercial product of great importance and of varied use. It is employed for bleaching and of late years rather extensively for purification of water. The latter use has given rise to much complaint, though in many cases the taste of the water is not due to the free element. The use as a war-gas was inaugurated by the Germans in April, 1915, at Ypres, at a point in the Allied line, which was held by French and British troops, the latter largely Canadians. Though the Allied intelligence service was advised about two weeks previously that the Germans were establishing apparatus for gas attack, this was not believed and the attack when made was most disastrous. Very soon, however, gas masks were supplied and chlorine as such is not now available for war purposes, but largely used in the preparation of asphyxiating and irritating substances. The extensive use of liquid chlorine for bleaching and water purifying has rendered necessary strict regulations for its transportation, a matter which has received the careful attention of the Interstate Commerce Commission.

At the forty-ninth meeting of the American Electrochemical Society (Chicago, April 22–24, 1926) two papers relating to the use and transportation of chlorine were read. The relations of the element as a war-gas were presented by Major-General Fries, Chief of the Chemical War Service, and the transportation of the liquefied substance was described by Messrs. Wells, Marey and Rowland, experts connected with establishments making the material. Chlorine is transported largely in steel cylinders, but tank cars holding slightly less than 12,000 litres (421 cu. ft.) are also used. The precautions of shipment and the construction of valves and other accessories are described in detail. Major-General Fries gives an account of some of the important war-gases now recognized as efficient, and states that this method of warfare is less inhuman than the older methods. Many gassed persons recover completely under proper treatment.

H. L.

Dr. Mann Juvenile Lecture. (*Jour. Roy. Soc. Arts*, Jan. 15, 1926.)—After the account of a lecture on "France in North Africa" and preceding the text of a Cantor lecture on "Coal Ash and Clean Coal," there is printed a report of this juvenile lecture on "Alice in Wonderland at the Breakfast Table," presented by no other than Henry E. Armstrong, F.R.S. Alice came in through a looking-glass and, finding the Duchess, the March Hare, the Mad Hatter and Brer Rabbit at the morning meal, "proceeded to discuss in Wonderland fashion such common objects of the breakfast table as sugar, and sausages, as well as cabbages, beetroots, and in general such questions as

"Why the sea is boiling hot
And whether pigs have wings."

They did, in fact, carry out on the stage an experimental enquiry as to why ice cream freezes at 32° F. and at 0° C.

Professor Armstrong, in a doctor's gown and a couple of hoods, presided over the company in the character of an academic Father Christmas—a part which he explained had been offered to Sir Oliver Lodge and declined by him with contumely.

For some reason the British pay more attention to presenting scientific matters to young people in lecture form than do we in the United States. The Christmas lectures to children at the Royal Institution and similar lectures given in connection with the meetings of the British Association for the Advancement of Science are fine specimens of this type of elementary exposition. C. Vernon Boys on "Soap Bubbles" and John Perry on "Spinning Tops" are illustrations of such lectures at their best. When Sir William Bragg, at the time of his presence at The Franklin Institute Centenary, held two thousand boys in the Northeast High School of Philadelphia interested in his presentation of some problems on the frontier of crystallography, he provided an example well worthy of imitation by other eminent men of science.

G. F. S.

THE ERECTION OF THE SUSPENDED STRUCTURE OF THE DELAWARE RIVER BRIDGE.*

BY

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THE suspended structure of the Delaware River Bridge is that part supported by the main cables. It consists of a main span of 1750 feet and two side spans of 716 feet 8 inches. There are two stiffening trusses, hinged at the towers, into which the floor system is framed. The trusses are of the through Warren type with verticals. They are 28 feet deep with panel lengths of 20 feet 6 inches in the main span and 20 feet 8 inches in the side spans. A section of the structure is shown in Fig. 1. The vehicular roadway will be paved with a 2½-inch asphalt wearing surface supported by a 6-inch reinforced concrete slab. At this time the steel work of the trusses and the roadway slab have been completed.

The steel for the suspended structure was furnished and erected by the American Bridge Company and the roadway slab was laid by F. J. Boas. The first piece of steel was erected on March 3, 1925, and the last section of slab was poured on December 8th of the same year. In the nine months that elapsed between these dates, 18,500 tons of steel were erected and 2183 lineal feet of roadway slab were poured.

The erection was divided into two distinct phases: Hanging the steel on the cables and adjusting the members for riveting. The distribution of labor between these two phases was about 16,000-man days to the first phase and 20,000-man days to the second.

The slab construction was essentially part of the second phase, since it was written into the contracts that the slab program should be laid out primarily to assist in the adjustment and riveting of the structure. The zeal of the steel contractor to erect record tonnage increased the cost of the second phase, since the possibility of regulating the erection to assist in the

* Presented at a joint meeting of The Franklin Institute and the Philadelphia Section, American Society of Civil Engineers, held Thursday, March 4, 1926.

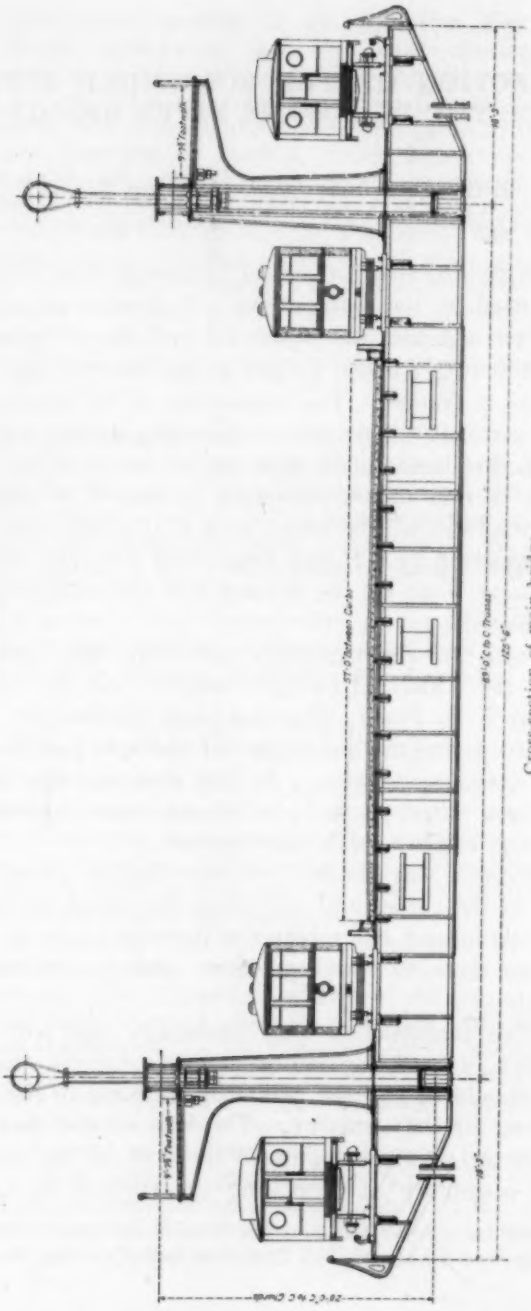


FIG. 1.

Cross-section of main bridge.

adjustment and riveting was neglected. A suitable regulation of erection would have obviated the use of such surcharges as were necessary when this opportunity had been neglected.

ERECTION.

The erection scheme was fundamentally the same as used on other suspension bridges, and was based on the fact that a hanging rope is an ideal support for uniform loads. Following this principle, the dead load of the structure is applied in successive small increments and the erection of any increment is completed before another is begun. Large concentrations should be avoided and it follows that the erection equipment should be as light as possible. Heavy travellers are emphatically out of place, since the distortion of the unstiffened cable under such concentrations is sufficient to prevent the connection of members that must be connected before the erection can proceed. Such equipment may also cause permanent damage to the cables by slipping the different strands through the tower saddles and by shifting the cable bands.

For these reasons, the steel was erected in successive trips or passes of the travellers, adding at each pass increments of the dead load. The first pass contained only the necessary material to support the travellers and was the heaviest pass. The weight of this pass was about 7000 pounds per lineal foot of bridge, which was one and one-half times the weight of the cables and their appurtenances.

Under the cables alone the towers were deflected 15 inches shoreward and in order to decrease this unbalanced pull the first steel was erected in the main span. It was placed with 106-foot booms attached to the main towers and consisted of three panels of first pass steel. This material formed the platforms on which the travellers were assembled. After the main-span travellers were assembled, they erected several panels of steel before operations were begun in the side spans.

The erection equipment consisted of four travellers, two in the main span and one in each side span. They were identical except for the hoisting engines with which they were equipped. They were of the conventional double-boom type, the booms being 70 feet long and of 45 tons' capacity. The travellers moved on two lines of railway track laid with 85-pound rail. The

tracks were spaced 42-foot centres and each was carried by three lines of roadway stringers. The travellers were 42 feet 5 inches long, centre to centre of trucks. This distance was slightly longer than two panels of bridge, so that the traveller reactions were at no time on two adjacent panels. Each boom of the travellers was operated by an electric hoist of from 55 to 100 h.p. on 500-volt d.c. All engines were of the two-drum type and the booms were swung with hemp lines from "niggerheads." The travellers weighed about 135 tons.

The economy of two-drum engines is doubtful on erection of this type where a large part of the tonnage consists of light pieces. With the two-drum type of engine only one load line is available and it must be reaved in as many parts as are necessary to lift the heaviest piece. Being of so many parts, the falls moves very slowly. A fast running falls for raising light loads could be provided when three-drum engines are used. The use of such a falls would have saved much time, especially in erecting the first pass material where almost half the tonnage was composed of pieces weighing less than 4 tons. However, this saving might be offset by the large investment necessary should the three-drum engines have to be purchased when the two-drum types were already on hand. The power for operating the travellers was furnished by synchronized motor generator sets located in the power plants near each anchorage.

Material was delivered to the site in cars on carfloats except that shipped from the yard on trucks for that part of the structure over Delaware Avenue in Philadelphia. In general, the steel for one first pass move was carried on one carfloat. After the first few moves, this amount of steel was erected in one day, the traveller moved forward, and made ready to erect the next carfloat load the following morning. For succeeding passes steel enough for several days' work was carried on the float at one time. As the erection proceeded, the float was moved to a point under the traveller and moored in that position. A tug-boat remained in attendance throughout the day to move and turn the craft when convenient for the erectors.

As the travellers moved out from the towers with the first pass erection in the main span, the bottom chord assumed a curve concave upward instead of convex, which is the finished shape of the chord. Because of this distortion it was impossible to

bolt any of the bottom chord splices during the erection of this pass. In fact, it was often difficult to connect the stringers necessary to support the travellers as they moved ahead. This difficulty was due to the heavy concentration of the traveller at the point and in some cases it was necessary to move the traveller back before the stringers could be bolted and the erection proceed. After the travellers had passed the quarter points of the main span, the bottom chord became convex downward and further

FIG. 2.

First pass erection showing reverse curve in bottom chord.

erection altered the curve in the direction of the dead-load parabola. However, at no time during the first pass erection did the bottom chord lie in a smooth curve, but showed the profile of the structure as a series of stiff sections two panels long. This is clearly shown in the deflection curve taken at the time of closure.

In detail, the method of erecting the first pass steel was first to attach the verticals to the suspenders; then the bottom chords to the verticals; and then fill in the first pass floor material. The verticals were connected to the suspenders by slipping the suspender sockets under the shelf angles on the member and holding the sockets in place with the slotted shims as shown in the figure. The shims were secured by bolts extending through the

FIG. 3.

Attaching vertical to the suspender.

shelf angles and the shim packs. The pack of shims varied from 2½ to 5 inches to make up the inaccuracies in fabrication of the trusses and in cutting the suspenders. It may be said in this connection that the calculation of suspender length and the attendant operation of cutting was accomplished with great success. The determination of length involved not only extensive calculation, but also sound judgment in the interpretation of laboratory tests in a pioneer field. The modulus of the rope

FIG. 4.

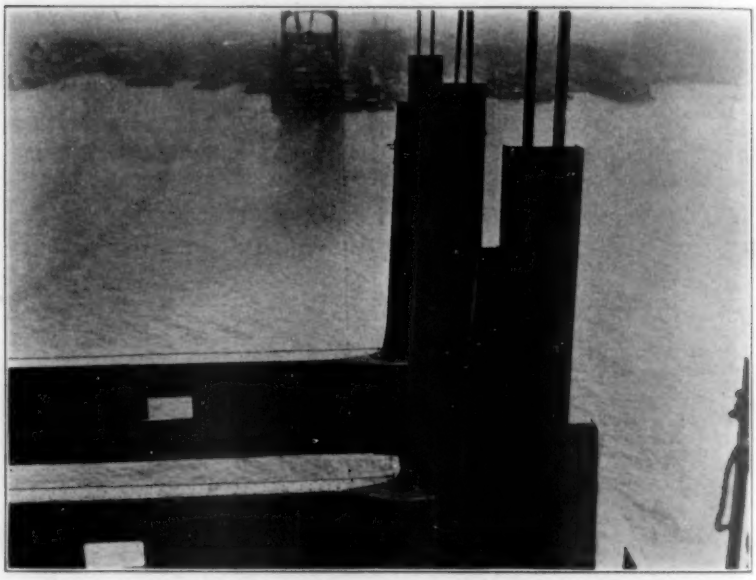
Erecting floor beam, bottom chord in position.

varied because part of it had been used in the foot-bridges and some of the inelastic stretch had been removed and the determination of these moduli required careful tests and sound interpretation of the results. The results obtained in the finished structure could not have been better had the dead-load conditions been simulated for measurement and cutting. The field operation of measuring and cutting was accomplished with such precision that the 596 ropes are correct in length and no readjustment of shims was necessary after erection.

The procedure of first pass erection was varied in the centre of the main span and the top chord and diagonals were added to the other material since it was found impractical to erect the top chord after the other material had been placed on account of the insufficient clearance between the top chord and the cable.

The closure was made at mid-span on May 7, 1925. At this time the main-span erection had so far outdistanced that of the side spans that the towers were deflected toward the river and the centre of the span was about 2 feet below its full dead-load position. This caused the two halves of the main span to drift together and made the opening for the top chord closure 5 inches short. The two halves had to be pulled apart before the closing chord would enter. This was accomplished by pulling with ratchets at the

FIG. 5.

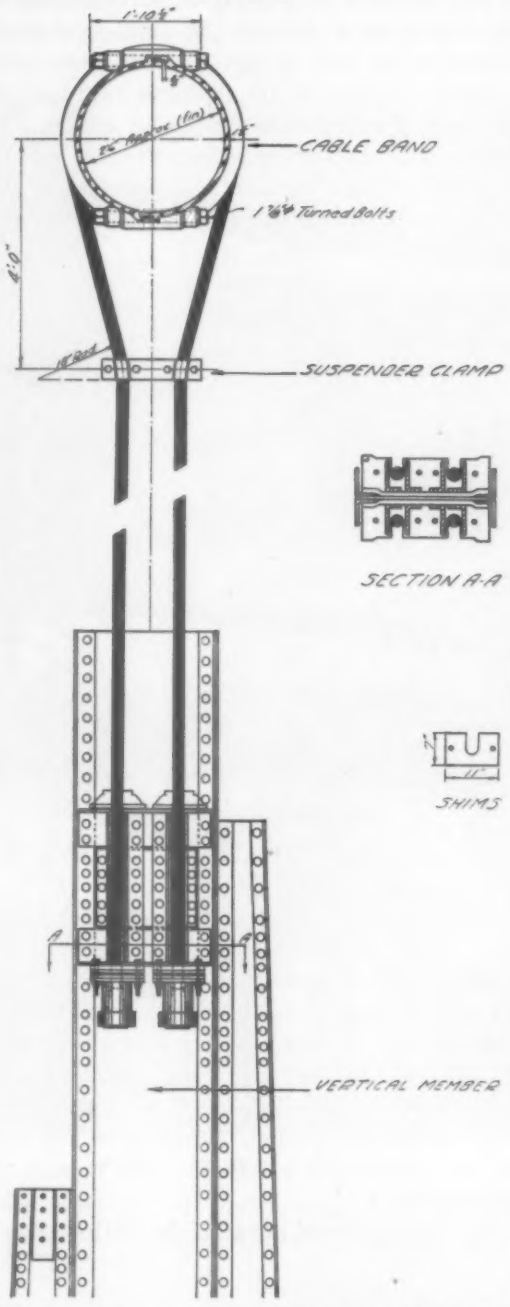


Suspender connection to verticals.

towers and by jacking at the point of closure. The operation was performed without difficulty and both trusses were closed on the same day.

During the return trip of the travellers to the towers, the second pass was erected. It consisted of the diagonals and was completed in two days. Starting out again toward the centre, the main-span travellers began the erection of the top chord as the third pass. In the meantime, the side-span travellers moved back from the centres of the side spans where they had been erecting first pass steel and started erecting top chords, diagonals and whatever other material was on hand. This was done because

FIG. 6.



Detail of suspender connection to vertical, showing shims and sockets.

further first pass material had not been delivered and weight was needed in the side spans to balance the main-span erection.

The erection of the top chords was the most laborious part of the first phase. It involved the disconnection and reconnection of 298 suspenders or every other suspender group on the bridge. The top chord was shipped in two-panel lengths, the splice falling at the panel point. The suspenders at the middle point of the chord section had to be disconnected before the chord section could

FIG. 7.

Conditions at first pass closure.

be placed and then passed through a hole in the cover plate and reconnected after the chord was in position. The suspenders were handled with a hydraulic pulling jack device. It consisted of four 15-ton jacks fastened to pins through the gusset plates on the bottom chord. A special casting gripped the bottom ridge of the socket and was connected to the jack with a short length of wire rope. When the suspenders were to be released, the pressure was applied to the jacks which pulled the sockets down by stretching the suspenders and cleared the shims. This enabled the men to remove the shims and pull the suspenders clear of the shelf angles when the jacks were released. The reverse process was used for reconnection.

By erecting the top chord outward from the towers, two clos-

ures were necessary, since part of the chord had been erected during the first pass. Due to the local deflection under the

FIG. 8.

Closing chord in place.

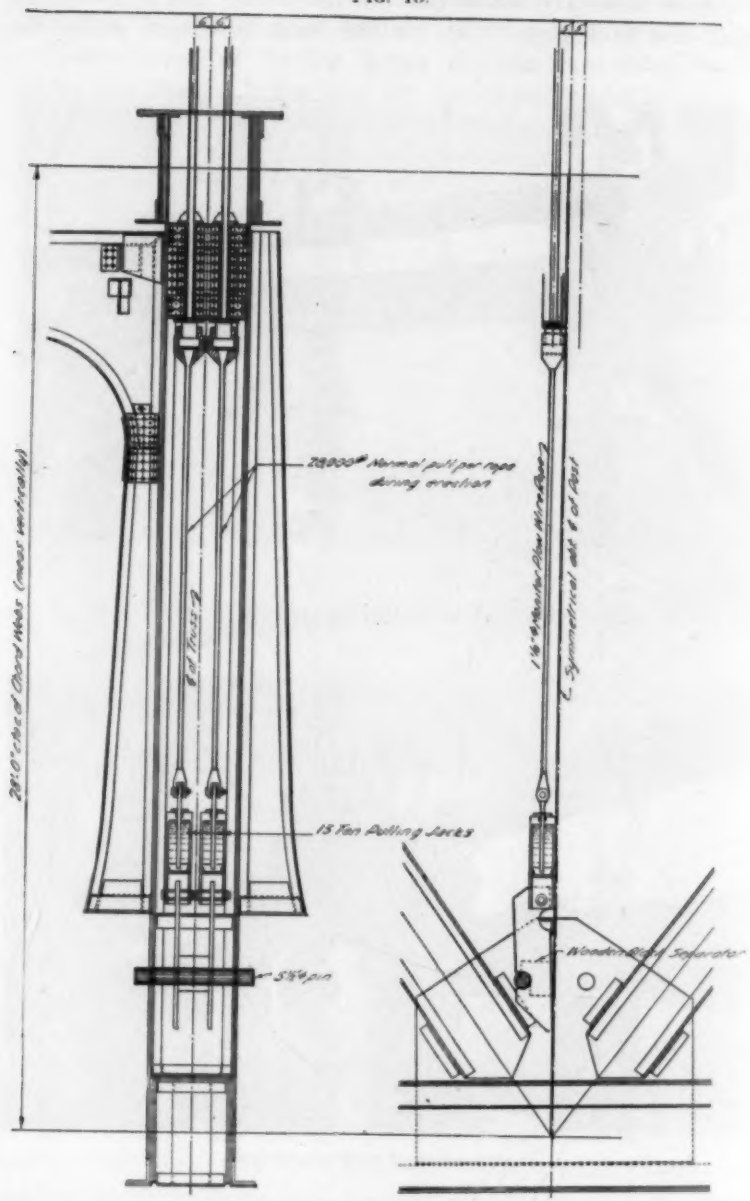
travellers when standing at the closure points, the opening was

FIG. 9.

Jacking top chord to obtain clearance.

too small for entering the closing section. The chord sections were then lashed to the cable directly above the opening and the

FIG. 10.


28000# Normal pull per rope during erection
8 at Thrust
15 Ton Pulling Jacks
5½ pin
1 6" Wooden Pier Wire Rope
2 Symmetrical abt 4 at Pier
Wooden Body Separator

Details of suspender rope-jacking device.

travellers moved back to the towers. With the travellers removed from the vicinity, the local deflection vanished and the closing chords were lowered into position.

The remaining passes in the main span erected the outside track sidewalks, and floor material as received. During these passes the diagonals were connected whenever drift pins could be driven in the holes of the connection. In general, this was accomplished with the traveller standing in the vicinity, since its

FIG. 11.

Erecting top chords, showing two closures in main span.

weight was necessary to bring that portion of the span down to its approximate dead-load position. The side-span travellers in the meantime were moving toward the anchorages and completing the erection of the side-span trusses. The diagonals in the side spans were connected by the same method as those in the main span. After the diagonals had been connected, the trusses acted as stiffening trusses for the first time.

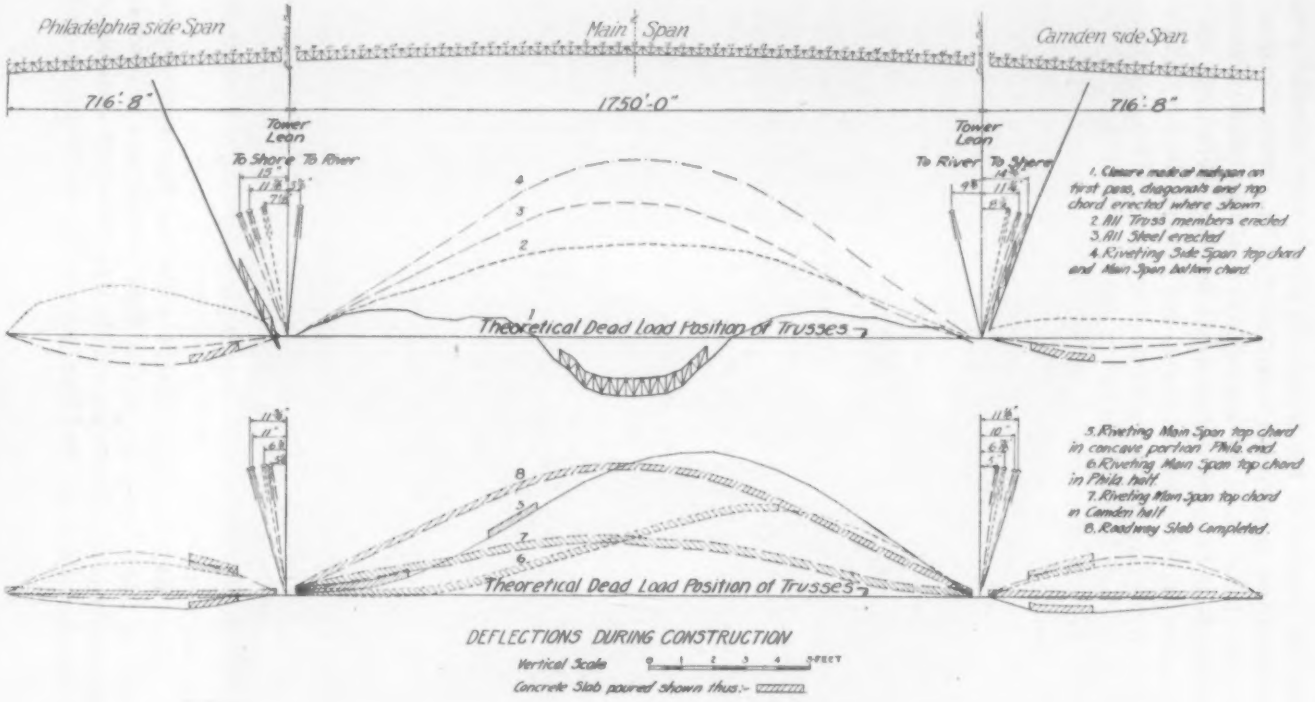
ADJUSTING AND RIVETING.

The second phase of the work was to bring the adjacent members of the trusses to their proper position with relation to one another and rivet them while in this position. Since this operation is peculiar to suspension bridges alone, a review of the attendant conditions and the premises of design is desirable. The trusses

in a suspension bridge carry no stress under full dead load at normal temperature. This premise of design fixes the shape of the trusses for fabrication. Accordingly, the main-span trusses were built with their neutral axis forming a parabola with a 15-foot rise. The side-span trusses were laid out with their axes as straight lines on a 3½ per cent. grade. Before driving any field rivets in the truss connections, it was necessary that the members to be riveted be in the same relative position as they had been when the trusses were assembled in the shop during reaming. In this position, the milled ends of the chord sections were in bearing and the holes in the other connections were fair. From these conditions it follows that in order to rivet the diagonals, the trusses must be in their "no stress" position while chord splice riveting demanded that the chord be in compression so that the milled ends of the sections be in bearing. In order to establish the "no stress" position of the trusses, the effect of whatever dead load that was not in place must be simulated. To bring the top chords in compression, the effect of live load must be attained in addition. About 6000 pounds per lineal foot of bridge, or 23 per cent. of the entire dead load, is made up of paving, sidewalks, ornamental iron, tracks, conduits, etc. Without this load in place, the cables have less than the full dead-load sag and the trusses are in negative bending. To realize the full dead-load condition over the entire structure at one time would have required about 7000 tons of surcharge, the cost of which would have been prohibitive. Accordingly, the expedient of partial loadings was adopted and the roadway slab used as surcharge.

This expedient took advantage of the delicate balance of the structure whereby the effect of loading one span without a corresponding load in the others was to bring that span down to, or below, its "no stress" condition. Small sections of slab were poured in the side spans before any was poured in the main span. This operation brought the side spans down to one foot below and raised the main span to a point 5½ feet above the "no stress" position. The compression in the top chords of the side spans and in the bottom chords in the main span was sufficient to bring the joints to perfect bearing and they were riveted. After this riveting had been done, the concreting of the main-span slab was begun near the Philadelphia tower. With the addition of this

FIG. 12.


Philadelphia side Span
Main Span
Camden side Span
716'-8"
1750'-0"
716'-8"
Tower Lean
To Shore To River
Tower Lean
To River To Shore
1. Closure central main span on first pass, diagonals and top chord erected where shown.
2. All Truss members erected.
3. All Steel erected.
4. Riveting Side Span top chord and Main Span bottom chord.
Theoretical Dead Load Position of Trusses
5. Riveting Main Span top chord in concave portion Phila. end.
6. Riveting Main Span top chord in Phila. half.
7. Riveting Main Span top chord in Camden half.
8. Roadway Slab Completed.
Theoretical Dead Load Position of Trusses
DEFLECTIONS DURING CONSTRUCTION
Vertical Scale 0 1 2 3 4 5 FEET
Concrete Slab poured shown thus:-

slab, the portion of the trusses in the vicinity assumed a radius of curvature greater than that of the "no stress" curve and the top chords came to bearing for riveting. Slab construction was continued toward the centre of the span, deforming the truss as shown on the deflection curves and keeping the top chord in the position for riveting as the work progressed. This operation was successful until the centre of the span was reached. From this point on, the effect of the slab as partial loading was lost, but by pouring the remainder of the span, the centre was brought down to within 18 inches of its "no stress" position. With the truss in this position it was possible to bring the remaining top chord splices to bearing by the use of small surcharges. Two surcharges were used, one a 100-ton load of pig iron on trucks and the other 200 tons of reinforcing steel which was moved by hand when necessary. By manipulating these surcharges to bring portions of the top chord in compression at a time, the remaining splices were riveted. The closing of one splice was aided by heat. In this case one web of the splice had been closed and riveted during the day, but the other web had opened during a temperature drop over night. The open web was heated by oil-burning flame throwers which caused the open web to expand and brought the ends to bearing. Slab construction was suspended two weeks during the manipulation of these surcharges.

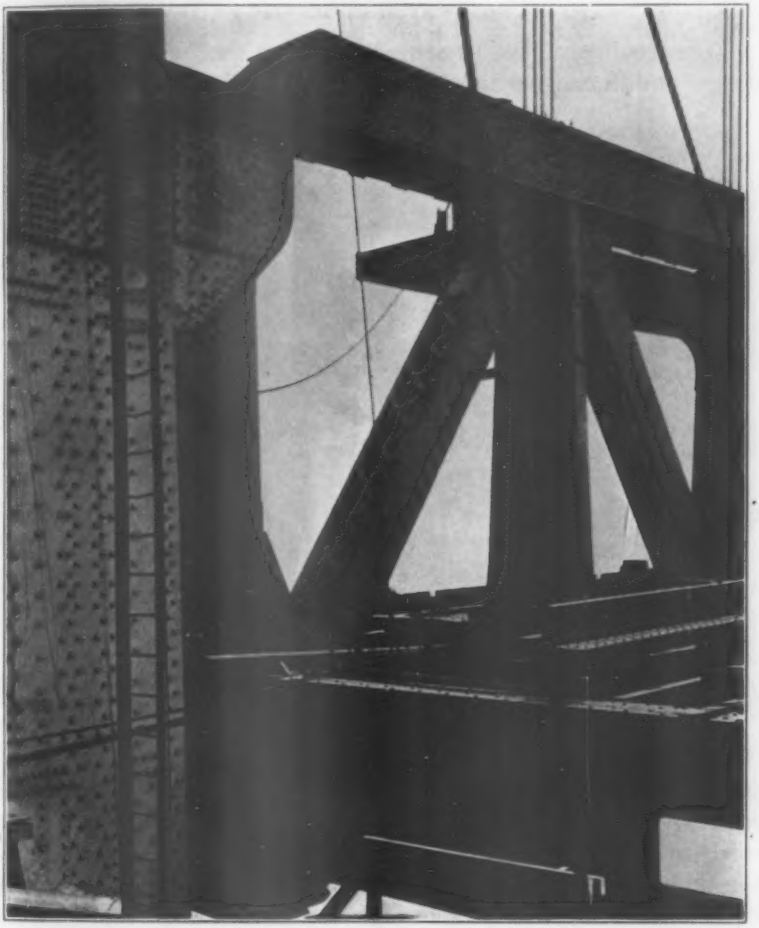
During this operation the side spans were 14 inches above their "no stress" position and the bottom chords in compression. The compression was sufficient to force the milled joints to bearing and they were riveted. Of the methods used in closing chord splices, that adopted for one especially recalcitrant top-chord splice in the Camden side span is of special interest. This splice had not closed with the compression that had brought the other splices in the span to bearing. To avoid moving the surcharges to this point the suspenders in the vicinity were released. This made the structure self-supporting and put additional compression in the chord. With this compression it came to bearing and was riveted. With all the chord splices riveted the concreting was resumed and the slab completed.

The diagonals were riveted whenever the truss approached its "no stress" position. The floor system was riveted during all positions of the trusses. This was allowable since the stress expansion joints in the floor prevent its participation in truss

stresses. Stress expansion is provided every two panels of bridge in the floor system. The stringers are free to move with respect to the rest of the structure at these points.

The ends of the suspended spans are connected to the towers

FIG. 13.



Truss connection to towers.

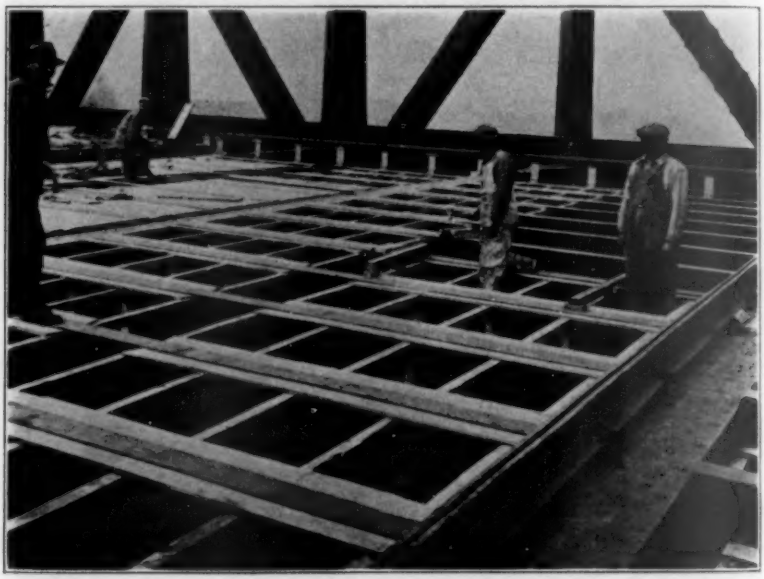
and anchorages with 16-inch pins. These pin connections carry only live-load reactions. Without the full dead load in place, the suspenders had not received enough stretch to bring the span down so that the pin-holes were matched. It was desirable to

drive these pins before the travellers were dismantled. In order to match the pin-holes the travellers were moved to the ends of the spans and the concentration of this equipment was sufficient to stretch the suspenders the required amount and the pins were slipped into position without difficulty.

ROADWAY SLAB DETAILS.

The roadway slab presented some unique features both in design and in construction. It is 6 inches deep, reinforced trans-

FIG. 14.

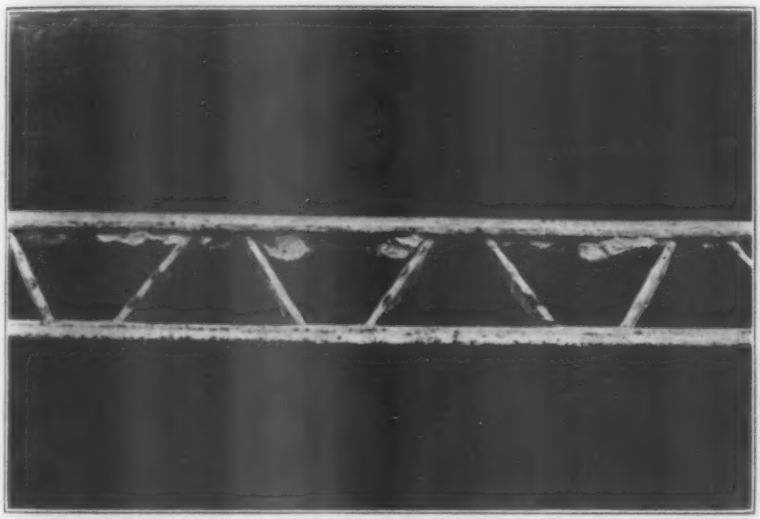


Slab forms and stress expansion joints.

versely with special steel reinforcing trusses and longitudinally with ½-inch deformed bars. The slab was laid on top of the roadway stringers. It was divided into sections two panels in length by the stress expansion joints. These joints consist of two 8-inch by 3½-inch bulb angles, back to back, that dam the concrete on either side of the joint. The bulb angles are anchored into the concrete with anchor bolts and fastened securely to the stringers on one side of the joint and to the floor beam on the other. The nominal distance between the backs of the bulb angles is ½ inch.

The reinforcing trusses are an especially interesting item as they are not only a new departure in roadway reinforcing, but are easier to place than the ordinary systems of reinforcing. The trusses are 4½ inches deep out to out of chords. The chords are rolled sections of U-shape which are connected to the flat web members by gripping with great pressure. The connection made in this manner is sufficiently strong to develop the web. The trusses were supplied by the Rivet Grip Steel Company, of Cleve-

FIG. 15.



Slab reinforcing truss removed from test slab.

land, Ohio. This contractor delivered them to the site and they were placed by the slab contractor.

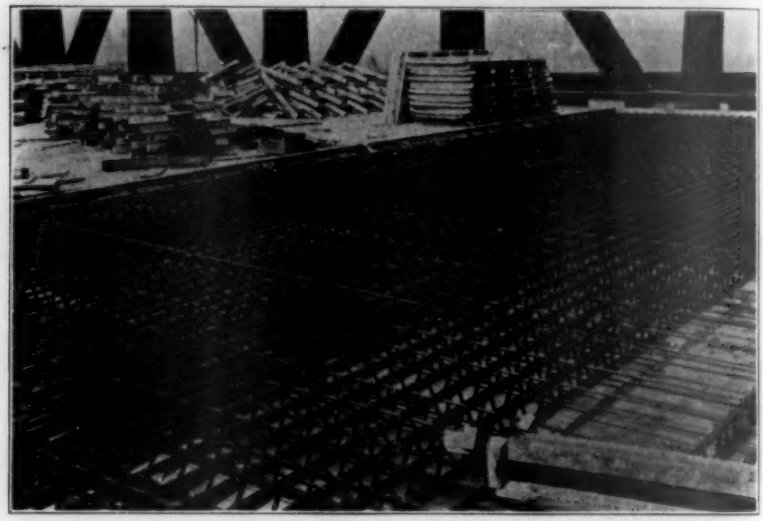
The trusses were held in position during pouring by special steel chairs that rested on the roadway stringers. These chairs not only held the trusses to their correct elevation with respect to the forms, but also spaced them to the correct spacing of six inches. The network of reinforcing was completed by the longitudinal bars. This system of reinforcing was very rigid and was not displaced in the pouring operation.

The forms for the slab were of simple design and were made of wood. They were used over again several times owing to the ease with which they could be stripped. A scaffold one panel long and the complete width of the roadway was used in stripping.

The concrete was mixed at a central mixing plant located near the Philadelphia tower and hoisted to the bridge deck. It was distributed along the structure with one-yard dump-body Ford trucks. Until the slab was constructed, these trucks were carried on a plank roadway. The maximum day's run was five expansion sections or over 200 feet of 57-foot slab.

Great care was used to secure a bottom surface that was free from honeycomb or voids. Two pneumatic, vibrating, rubber-shod

FIG. 16.



Slab reinforcing in place.

hammers were used to vibrate the forms and the reinforcing steel while the concrete was being placed. In addition to this precaution a workable mixture of concrete was used with a high cement content. These precautions were well worth the effort, as the under-surface of the slab is very smooth and free from voids. It is also felt that vibrating the forms has produced a dense concrete that will stand the test of time and hard usage. The specifications for the nominal 1 : 1½ : 3 concrete used in the slab call for a strength of 2500 pounds per square inch and the average strength of the test cylinders made in the field is 2695 pounds per square inch.

In conclusion, it is fitting to give great credit to the excellent spirit of coöperation evidenced by the different contractors. Although forced to carry on different operations in the same limited space, they adopted a policy of give and take for the good

FIG. 17.

General view with slab complete.

of the work. It was only due to this splendid spirit of coöperation with each other and with the engineers of the Delaware River Bridge Joint Commission that the same high standard of work that has been done on the rest of the structure was maintained.

CABLE CALCULATIONS FOR THE DELAWARE RIVER BRIDGE.*

BY

G. M. RAPP, Jun. M. Am. Soc. C. E.

Assistant Engineer, Delaware River Bridge Joint Commission.

THE main cables have received a degree of popular interest and professional attention that has probably not been accorded any other single element of the Delaware River Bridge.

And perhaps no other part of the bridge so well deserves such interest and attention. The two cables compose the main load-carrying member of the bridge and exert their influence over all parts of the structure. The design of the towers and anchorages which support and anchor them depends upon our knowledge of their behavior, while the stiffening trusses, in performing their important functions of limiting distortion and distributing load, follow the cables closely in all movements, and a rational analysis of the former necessitates a full knowledge of their action. It is also apparent at once that the erection of so important a member of a load-carrying system entails additional calculations of considerable refinement. Therefore, it will be proper to treat the subject-matter with reference to (1) tower analysis, (2) stiffening truss analysis, and (3) erection. In the sense used here analysis means that phase of design wherein we treat only of the calculations necessary to determine the forces, stresses and motions in the various parts of the bridge. It should by no means be construed to embrace those features of design that involve planning, choice of character size and shape, proportioning of members and parts, and the many other duties in the realm of design.

The cables themselves have been so frequently described that I shall only briefly outline their composition, physical qualities and geometry. Two in number, they are each made up of 18,666 parallel steel wires bound tightly together into a compact circular cylinder thirty inches in diameter. Each wire is 0.196 inch in diameter over galvanizing, has a specified minimum ultimate strength of 215,000 lb. per square inch, a specified minimum yield point of 144,000 lb. per square inch, and a coefficient of

* Presented at a joint meeting of The Franklin Institute and the Philadelphia Section, American Society of Civil Engineers, held Thursday, March 4, 1926.

elasticity of about 27,000,000 lb. per square inch. Each cable is 3550 feet long from centre to centre of strand-shoes at the anchorages and is held immovably fixed by friction to the towers and anchorage cable bents which support them. The normal sag between towers in the centre span is 200 feet; the normal sag from the chord to the table curve in the side spans is 37 feet. In round numbers the two cables contain over 25,000 miles of wire, or enough to extend once around the earth's equator, have an ultimate or proof load combined of 120,000 tons, and a total elastic stretch from no load to fully loaded condition of about 15 feet.

Let us now view the action of the bridge under loading conditions. With no live load on the structure and with an assumed normal temperature the entire dead load is carried by the cables. The trusses remain suspended free in an unstressed condition. The cables hang in their natural curves, parabolas, for this dead load, which is assumed distributed uniformly over each span. In this state the cables, towers and trusses form an elastic system in equilibrium. With the advent of live load or a change in temperature, or both, the cables change their length and shape, the towers bend, and the trusses deflect to accommodate these changes and the entire system takes a new position of equilibrium. The cables and trusses each then take a share of the live load, and the partition of this load depends upon the relative stiffness of the two. The trusses, acting as beams, carry their proportionate amount of the load to the supports at the towers and anchorages and distribute the remainder of the load to the cables. It is largely this redundancy of load partition or static indeterminateness that renders the problem of the suspension bridge difficult for solution.

In studying the effect of loads on the structure it must be borne in mind that no single span is complete by itself; that is, a load on any span is reflected in the other two, or, stated in other words, loads on any span have their counterpart in loads necessary to balance them on the other two spans. This adjustment of load between spans is simply the balancing of three equilibrium polygons. To illustrate let us take the case of a uniform live load in only the centre span: The part of this load that goes to the cables increases the stress in them and they therefore stretch or increase their length, which causes in turn an increase in their centre sag. This is accompanied by a displacement of the tower

tops inward, or towards each other, which again increases the sag. These increments in sag, on the other hand, cause a decrease in cable stress with a consequent shortening of length, and decrease in sag. There is a balance between these two effects. This is not all, however, for the balancing of the horizontal components of cable stress at the towers requires that the stress in the side-span cables should also increase. To do this they must change their shape or find load; in effect they do both of these things, for they flatten their curves by increasing their span (that is, by letting the towers over) and in so doing they pull up on the trusses, which, by reason of their resistance to bending, supply a load. The whole system ultimately comes to rest with equilibrium between three cable polygons, on the one hand, and three elastic beams or trusses on the other. Similar reasoning will apply to live loads on the other spans, and also to temperature changes from normal, which, in shortening or lengthening the cables, act as so much equivalent live load. It is obvious that the shorter the span of the trusses, the stiffer they will be and the more resistance they will offer to motion, other things being equal; and so it is not at all surprising to find that the side-span trusses carry approximately three-quarters of the live load on the span and that the centre-span trusses carry only about one-twentieth of the live load on that span. This, interpreted in another way, means that loads on the side spans will have relatively little effect on cable stress compared with those on the centre span. As a last word on the operation of the bridge under load, it is helpful in visualizing load effects to compare the bridge with a seesaw or lever, in which the side and main spans are the two arms and the tower the fulcrum: That is, loads on the main span cause a downward deflection there and an upward deflection in the side spans, and similarly, loads on the side spans deflect those spans downward and the centre one upward.

A discussion of cable calculations in relation to stiffening truss and tower analysis can hardly be considered complete without some mention of elastic deflections. Any statements that are made concerning these must, however, first be qualified and the results judged accordingly. In the first place, these motions are based upon certain assumptions of loading and temperature. They not only assume the greatest possible density of traffic and

extremes of temperature, combined with segregation of load, but they also require these several conditions to be simultaneous and in the proper relation to each other to produce maximum effects. In so much and so far they are no different from the usual loading criteria for ordinary short-span bridges. But when the spans are long the probability factor becomes important. Like the design of flood control works, dams, or sewers, the maximum conjecturable or estimated peak loads must be cared for and the structure made safe to withstand them. Here the analogy ends, for in a structure as flexible as the suspension bridge the changes in form that accompany these peak loads are relatively large, and their meaning may be misinterpreted and their importance misunderstood. In some cases of the suspension bridge these large distortions may be for the most part only of academic interest, but in other cases they may attain prime importance. In connection with the tower, large deflections cannot be other than of great importance; in speaking of the stiffening trusses, they may or may not be so considered, depending upon the character of the traffic to be carried.

To illustrate—the maximum downward deflection in the centre span would be obtained when that span only is loaded from end to end, with both high-speed car-tracks and surface car-tracks fully occupied with crowded trains and cars, the fifty-seven-foot roadway jammed with heavy vehicles, and the two ten-foot footwalks crowded with people; and furthermore, the temperature would have to be 110° F. at the time. It can be seen that the probability of such dense, segregated loading is remote and could be obtained only under most unusual conditions. Such, however, is the character of our very aptly termed "congested live load" of 12,000 lb. per foot of bridge. This loading was used in the design of the cables. It was also used in the design of the stiffening trusses and towers.

CABLE DESIGN.

Among the first calculations that must be made are those for determining the maximum pull in the cables. This must be done before either anchorages, towers or trusses can be designed. It may seem a bit peculiar, but it is nevertheless true that the design of the cables themselves, that is, the determination of the required

cable area, is probably the most simple problem of the suspension bridge. The largest cable stress requires the bridge to be fully loaded from end to end at the time of lowest temperature, as this is the temperature which gives the smallest sag. The greatest cable stress is found at the towers where the secant of inclination is the largest. This stress divided by the assumed unit stress of 72,000 lb. per square inch gives the required area of cable. It would seem necessary in some cases to investigate the possibility of a slightly different loading giving a secant at the tower enough greater than the one for this case of full load to more than offset the effect of a decreased stress due to the change of load. In general the cables are considered perfectly flexible, that is, having no resistance to bending, but at and near the towers there are certain secondary stresses. These were investigated and allowed for in the design. Extensive tests were made upon a section of sample cable to establish empirical laws of bending and internal friction, or longitudinal shear. The data of these tests were interpreted by means of a very nice theory that had for an hypothesis the creation of a " zone of slip " within the cable.

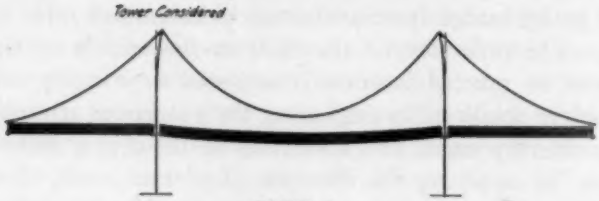
TOWER ANALYSIS.

In their relation to tower design, or more properly tower analysis, cable calculations play an important rôle. The main towers present the problem of cantilever beams each compelled to deflect definite amounts while acted upon by three forces: The vertical reactions from the cables, the unbalanced horizontal cable tension, and the truss reactions. These forces, together with the displacement of the tower top, all of which are prerequisite to tower design, are dependent upon an analysis of the cable for several critical loading conditions. It is beyond the scope of this paper to treat all these cases of loading, but it will suffice to deal with the general case somewhat in detail. The method of treatment is one of " trial and error " and was developed by Mr. Allston Dana, Assistant Engineer of Design, and extended and elaborated by the writer. It is believed to be rather unique in that it lends itself readily to a physical conception of the problem at every step and is entirely free from complex mathematical quantities. It is even applicable to slide rule analysis with some care and ingenuity. The method is based upon the principle

that the summation of all the changes in horizontal span length over the entire bridge from anchorage to anchorage must be equal to zero. The deflection of the cable at the middle of the main span from its normal position is assumed or roughly estimated for the given loading; by neglecting the suspender stretch, which is relatively very small, this also gives us the centre deflection of the truss; by applying the theorem of virtual work to the full truss with uniform load we are able to calculate its stiffness and express this in terms of intensity of load necessary to deflect it one foot at the middle of its span. We are thereby enabled to determine the amount of load carried by the truss system for the assumed deflection; the remainder of the live load on the span plus the whole of the dead load is carried by the cables. It is then a simple matter to figure the increase in cable stress, thence the cable elongation, bringing in temperature effect if necessary, and finally from this new derived cable length and new sag, the new horizontal cable span. So far the main span is treated as a unit by itself. The side spans are slightly more difficult to analyze: Here we have to find, first, the apportionment of live load between cable and truss, then the new cable sag, and finally the horizontal cable span that combined with this sag will give the required new length of cable found as in the case of the main span from stress and temperature changes. By balancing the H's at the tower the sag is expressed in terms of the main-span sag and the intensity of load on the cables in both spans; it may also be written as the sum of the original sag and the new side-span truss deflection, which latter is expressed as a function of the load on the span less the load on the cable. Here again the truss stiffness factor is calculated separately by means of virtual work. In these two equations we solve for two unknowns—the side-span cable sag and the side-span cable load. It only remains to find the horizontal span that is concurrent with this sag and with the length of cable calculated from stress and temperature elongation or shortening.

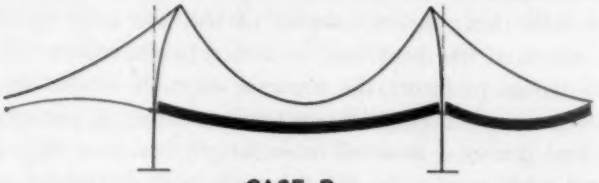
It is evident that the main span's gain in horizontal length must be the side span's loss, or, in other words, that the summation of all changes in length over the entire bridge must be equal to zero. Summing up in this way, we obtain a difference by which we fail to balance, and, applying this with some judg-

FIG. 1.



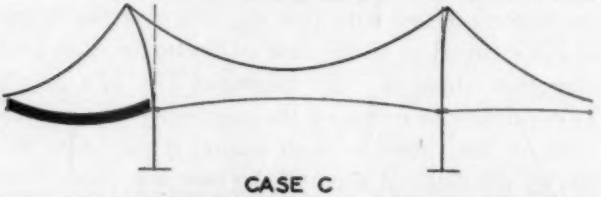
CASE A

Position of Load	Live Load	Dead Load	Temp	V	P	Defl.	Truss Reactions Side Span	Truss Reactions Main Span
Over all spans	12,000 #/ft	26,000 #/ft 115 27,000 #/ft 55	Lowest	34,700,000 #			+ 10,000 #	+ 6,000 #
" " "	6,000 #/ft	"	"	29,500,000 #			+ 12,000 #	



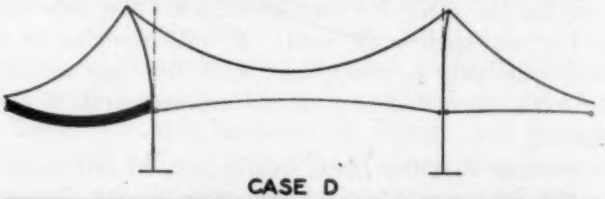
CASE B

Position of Load	Live Load	Dead Load	Temp	V	P	Defl.	Truss Reactions Side Span	Truss Reactions Main Span
Main for Side Span	12,000 #/ft	26,000 #/ft 115 27,000 #/ft 55	Highest	33,400,000 #	128,000 #	+18"	-1250,000 #	+ 180,000 #
" " " "	6,000 #/ft	"	"	28,700,000 #	80,000 #	+ 9"	- 640,000 #	+ 110,000 #



CASE C

Position of Load	Live Load	Dead Load	Temp	V	P	Defl.	Truss Reactions Side Span	Truss Reactions Main Span
Over Main Side Span	12,000 #/ft	26,000 #/ft 115 27,000 #/ft 55	Lowest	24,900,000 #	212,000 #	-21"	+1430,000 #	- 80,000 #
" " " "	6,000 #/ft	"	"	24,900,000 #	142,000 #	-13"	+ 70,000 #	- 60,000 #



CASE D

Position of Load	Live Load	Dead Load	Temp	V	P	Defl.	Truss Reactions Side Span	Truss Reactions Main Span
Over Main Side Span	12,000 #/ft	26,000 #/ft 115 27,000 #/ft 55	Lowest	19,200,000 #	317,000 #	-26"	+1590,000 #	- 90,000 #
" " " "	6,000 #/ft	"	"	18,800,000 #	223,000 #	-17"	+ 760,000 #	- 70,000 #

+ Deflections are toward the river.
− " " " " land
+ Reactions are upward
− " " downward.

ment to a reëstimate of centre sag we repeat the above process until a satisfactory balance of horizontal lengths is obtained.

In this method all cable curves, being the equilibrium curves for uniform loads, are parabolas, and the determination of their sags, lengths and spans is merely a matter of analytical geometry. When a balance is obtained we have not only the desired tower deflection, but also a complete knowledge of, or basis for, figuring all forces, motions and stresses throughout the bridge for this condition of loading. Other conditions of loading are analyzed in a similar way; Fig. 1 will show the cases treated in the tower design and the resulting forces and motions. It is interesting to note in passing that the forces, motions, and therefore the stresses for working live load are not one-half those for congested live load though these two loadings bear a 1 : 2 ratio. This lack of proportionality of loads and motions, which precludes the use of the common influence line and which renders all stress calculations more difficult, is simply the result of the flexibility of the bridge.

The writer does not wish to pass on without just a few words in reference to tower motions. The idea of these massive steel columns, seemingly so stiff and rigid, bending to and fro with the advent of loads on the structure, has been the cause of no little wonderment to the layman, the cause of considerable worry and doubt as to their safety on the part of the newspapers, and, at times, even the cause of some unrest to engineers. It is emphasized that such motions are provided for in the design, are even desirable from certain points of view, and constitute a true elastic behavior. It is true that for certain light erection loads the towers become surprisingly sensitive and show high deflections, but, although these large motions are more serious at the light loads, they may be easily anticipated by calculation and limited by a proper erection program. It is interesting to observe that the total maximum range of top deflection is 39 inches for congested load and 22 inches for working load.

A word must be said, too, of the effect of temperature or thermal load on the tower deflection. Briefly stated and without qualification, a change of one-half the total assumed temperature range (which is taken in this case as 110° F.) will produce a tower motion of three inches either way, that is towards the river for an increase or towards the land for a decrease in temperature.

STIFFENING TRUSS ANALYSIS.

First and last, the problem of the suspension bridge is an elastic one, and strictly speaking such a bridge might well be classed as a kinetic structure. Deflections are in the order of magnitude of feet instead of inches, as in the ordinary short-span bridges, and it is for this reason that the assumption always made in the latter that the lever arms of the forces do not change under load becomes invalid. When one considers this fact it is surprising, but nevertheless true, that most of the theories in existence and common use in the design of such structures calmly neglect this well-known fact and are based instead upon the assumption that the cable polygon remains undistorted under live load. They are called approximate theories and hold their popularity primarily because they are simpler in application and are free from the degree of mathematical intricacy found in the more exact theory. The effect of these approximate theories is to produce stiffening trusses less economical than those designed according to the more exact theory, and the extent to which they may prove uneconomical may vary from a negligible amount to as high as 40 per cent., depending upon whether or not the trusses are stiff enough to limit the polygon distortion to small amounts. In this bridge the exact theory was used and considerable saving in truss metal thereby affected.

This theory, which had its conception in the mind of Melan, was greatly elaborated, extended and placed in very attractive form by Mr. Leon S. Moisseiff, Engineer of Design, Delaware River Bridge Joint Commission. It has been used in the design of the Manhattan, Delaware River, and Poughkeepsie suspension bridges.

As in the case of the main towers an analysis of the stiffening trusses, that is, a determination of moments, shears, and deflections, depends upon certain cable calculations. The problem becomes one of finding the horizontal component of cable tension, H, for each condition of loading. Once this is found the rest is simple, and we may proceed directly to moments, shears and deflections without further difficulty. It has been shown above how this value of H can be found for the more simple conditions of uniform loading over the whole of any or all spans. Unfortunately the loading criteria for maximum moments and shears in the stiffening trusses are not generally susceptible to a similar

semi-geometrical method of treatment. In the former cases it will be noted that the cable curves were single parabolas corresponding to the uniform loads over the full length of each span; now, however, with loads covering only a part of the spans the line of the cable in such a span becomes two or sometimes three parabolas having common tangents at the points of change in load. This would render accurate solutions for lengths extremely difficult. Furthermore, the determination of the share of the loads carried by the cables would become very complicated. Such a method of attack, desirable as it may be, is not adaptable to the design of the trusses. We, therefore, have recourse to that most useful of all theorems of mechanics in dealing with indeterminate structures, namely, the theorem of work. By equating the work done by the suspender forces acting on the cable to the work done by the internal stresses in the cable integrated over the entire bridge, we are enabled to drive expressions for H in terms of the external loads on the bridge. Since the trusses coöperate with the cables in carrying live load, the equations for H are functions of the truss stiffness and the cable stiffness. The truss stiffness finds expression in terms of its moment of inertia and the modulus of elasticity of its metal, while the resistance of the cable to distortion, or its stiffness, is found in terms of its stress and its modulus of elasticity.

It is instructive in correlating cable and truss action to consider the effect of thermal load only on the structure. Take, for example, the case of a material increase in temperature with no live load on the structure: The cables of course will lengthen; the increase in length of the main-span cable will cause that cable to sag deeper, and the truss will therefore be lowered, or deflected, an equal amount. Inasmuch as any deflection of the truss must be caused by a load, this means that it is now carrying a share of the dead load, thus relieving the cables (which were supporting the full dead load) of a corresponding amount. This relief of load on the cables results in their shortening a certain amount in contra-action to the thermal elongations. There is a further reduction in their length, too, on account of the smaller stress caused by the increased sag. Meanwhile, however, the side-span cables have lengthened under the same temperature increase, and in doing so have allowed the towers to move toward the centre span. The effect of this is to again add to the centre-span sag,

which once more decreases the stress and length of the cables. There is a balance between these stress shortenings and thermal elongations. The solution sums up these effects over the entire bridge. When the balance sought is obtained, we find that the main span is deflected downward 1.8 feet for a temperature increase of one-half the range with no live load on the bridge. It is therefore carrying load, paradoxical though it may seem. Physically, this load can be no other than a portion of the dead load.

ERECTION.

Usually in the design of bridges over navigable waters the War Department imposes certain limitations at the outset. In our particular case the length of the centre span was limited to the clear distance between pier-head lines for a minimum, and the under-clearance had to be maintained at 135 feet above mean high water. These and other practical and economical considerations fixed the length of the side spans and the normal sag of the cables in the centre span. A combination of all these requirements determined the position of the bridge in a definite condition. We have constant reference to this condition in the design of the cables and suspended structure. It is the maximum dead-load condition, that is, the completed bridge with all its appurtenances, at a certain temperature. It is the basic position to which we refer all deflections and motions; and it is also the *status quo* from which all stresses are figured. For the erector in the field, it is his ultimate objective. In the calculation of the cables proper it is but the starting-point.

Theoretically the cables must be placed during their erection so that the bridge when completed will hang in or very near its correct position at a given temperature. Practically, we know that this ideal condition will probably only be approached. Certain phases of the problem are more or less beyond our control: Such as the modulus of elasticity of the completed cables, the non-uniform distribution of dead load, and the actual absolute span lengths. However, we know these physical constants fairly well and the effect of errors in them is usually small.

Let us progressively reverse the process of erection and see what part the cable plays therein. With the bridge in its final dead-load condition the cables hang in the shape of equilibrium polygons under the effect of the vertical suspender forces dis-

tributed uniformly along the span. As these forces are all evenly spaced and are of the same amount (an assumption which is very near the truth: The maximum variation from the average being less than 5 per cent.) the vertices of these equilibrium polygons will form in each case a definite mathematical curve—the common parabola. The exact length of the cables hanging in such curves can be determined. If now the weight of the entire suspended structure be removed and the cables left bare, they will shorten because of this relief of load and stress, and will hang in smooth curves under the sole effect of their own weight. The cables having a constant sectional area, this weight will be distributed uniformly along their own length, and they will therefore each have the configuration of a definite mathematical curve—the common catenary. This curve, like the parabola, gives an exact length of cable. The condition of catenary cable represents the cables in their position after they have been strung and before the dead load of the suspended structure has been hung upon them. In this state they are considerably higher at the centre of the spans than when they were fully loaded, and this amounts to seven feet in the main span for normal temperature, though it may be much more for lower temperatures. This serves to illustrate the magnitude of the motions that are involved, and which must be predetermined to a relatively fine degree.

The erection of a long-span suspension bridge, and this applies particularly to the cables, differs from that of its brothers, the cantilever and the arch, in that the procedure employed is standardized, the required accuracy is much less, the danger to both structure and life is a relative minimum, and the necessity of or occasion to compute any erection stresses is almost absent. Who would think of erecting the compression chord of an arch without very meticulously precalculating all erection loads and conditions, particularly at the time of closure? Yet in the corresponding tension chord of the suspension bridge, the cables, the very nature of the problem renders it impractical and totally unnecessary to carry on any such elaborate calculations for erection. Although a slight error in length of cable placed in any span will cause from two to five times this amount in vertical position of the roadway at the centre of the span, and may affect the final tower position, there is an opportunity in the determination of suspender lengths for adjusting this in the former

case, and the resulting stresses in the tower are small in the latter case. However, because the chance for large and intolerable errors is present, a close control of cable erection is pertinent and advisable. By "control of erection" I mean not that type necessary and paramount to all forms of control, field supervision and inspection, but the careful checking and precalculating of the cables in each and every stage of their construction. In carrying this out we were most fortunate in having contractors who coöperated with us in every phase of the work.

The calculations applied to erection may be grouped as (1) lengths of loaded cables, (2) lengths of unloaded cables, (3) the free strand, (4) sag-temperature-tower-motion charts, (5) closure figures, and (6) field check analysis.

The correct length of cable to be used in each of the spans is determined by setting definite points on the cables at prefigured elevations, and not by physically measuring off calculated lengths of wire. For convenience these points are taken at the mid-span of the cables. In doing this we are simply fixing a third point on a curve, the law of which gives us the relations between length, span and load. To begin with we do not know of the unloaded cables, either their exact span (that is, the position of the towers) or their length; but we do know these for the fully loaded cables, and furthermore, we know the change in load which occurs between the two conditions. If we can find a catenary cable in the centre span which has a stress differing from that of the loaded cable by an amount just sufficient to stretch the cable to its correct known length in the latter condition, we have determined the curve for the unloaded cable. Because the horizontal components of stress in the cables of all three spans are equal, we then have the change in stress in the side-span cables from which to figure their shortening and thence their unloaded length. Then it is a simple matter to calculate all the ordinates to the cables and the sags with which to set them. The actual calculations are somewhat complicated by the many details and corrections which have to be made.

It is important to note in the above deduction of catenary cable lengths that we have worked with changes in condition and not with one definite condition. The argument was often presented that, because of the lack of knowledge of the exact distance between towers, any cable lengths based upon these purely hypo-

thetical spans would be wrong, and that therefore the sags set from these lengths would be in error. Clearly, however, by working with only conditional differences, and by setting the cables in abstract elevation, we have largely obviated the likelihood of any such errors. If we have set the unloaded cables to an elevation based upon a hypothetical span, and the actual span is a little longer, then perforce there must have been more wire placed in that span, and, if the change in condition between dead load and unloaded cables remains constant (as figured), then there will be the extra needed length of wire in the parabolic cables to permit them to hang at the proper elevations when the bridge is completed.

The erector in the field does not set the cable as a unit, but as sixty-one individual units or strands. He must have a means of readily determining the correct elevation at which to set the first strand whatever the condition of span and temperature may be at the time. When the first strand has been set and adjusted, the remaining strands are set with reference to it; and so the question of how close the cable will come to its correct position revolves about the exactness and care with which the first strand is placed. Consequently a comprehensive sag chart which gives the elevations at mid-spans for varying tower deflections and a wide range of temperatures is prepared. The data for plotting these charts are computed as follows: For a given tower deflection and temperature we figure the changes in span and length of the cables, always referring to the changes in span to the maximum dead-load position in order to work only with differences in condition; other changes, such as the elongation of the main towers and cable bents which occur in passing from loaded to unloaded cables, are necessary refinements that have to be made. The change in length of cable due to the change in temperature from normal is applied to the normal catenary cable length, and a new length found. When we have the spans and lengths of the cables, we can find their sags by using the known laws of the catenary. It is then but a step to find the elevations required to plot the curves. A source of error likely to creep into the computation of these sag charts is the neglect of the elastic change in length of the cables due to stress. This can be corrected by a simple cut and try process.

It is convenient in dealing with the catenary curve to use

hyperbolic functions because these give very simple expressions for length and sag. It might be said here, too, that in all computations of cable lengths for erection purposes, exact equations should be used, as they are practically no more difficult to handle than the approximate ones, and the latter are likely to give errors too large.

While the strands are being strung they are supported in temporary sheaves on the towers and anchorage bents. They are then free to take a position of equilibrium dependent upon a balance of their total tensions on both sides of these sheaves. (Compare this with the equality of H's which is the criterion for the balancing of the strand after it has been laid in the saddle.) This is called the free strand. Computations have to be made in order to see whether or not it will foul the foot-bridges, and so interfere with the stringing of the wires. At once we know the total length of this strand from anchorage to anchorage: It is equal to the length of the unloaded cable less a correction. This correction provides for the difference in length along the centre line of the unloaded cable and that along the centre line of the particular strand in question. We also know the horizontal spans because the tower deflections are given. First, we assume a strand tension at the support, or sheave, and from this and from the known span and weight of the strand we compute its length in the centre span. One-half the difference between this length and the over-all length, which is known, will give us the length of strand in the side span, and, reversing the process of calculation used in the main span, we solve for the strand tension on the side span of the sheaves. A comparison of this tension with that assumed for the main-span portion of the strand will undoubtedly show a variation, and a second and possibly a third trial will be necessary before the required balance is obtained. Hyperbolic functions form very useful "mathematical tools" for solving this problem.

Reference has frequently been made to the fact that the horizontal component of cable tension, H, is constant and equal in the cables of all three spans. This assumption would be true if the cables were supported on frictionless rollers where they pass over the towers, or if the towers themselves had frictionless hinges at their base. As neither of these conditions prevails here, this assumption of equal H's is only approximate. The towers offer

considerable resistance to the horizontal motions sought by the cables while reaching a balance under load. These two systems, the cables and the towers, will resist any horizontal motions imparted to them in proportion to their relative stiffness. In stiffening truss analysis and in tower motion determination, where the cables are fully loaded, the effect of this differential H is small and fortunately it may be neglected. Indeed, by neglecting this additional force we are on the safe side. In the analysis of the unloaded, or the bare cable, it cannot be ignored, for it is relatively large in comparison with the forces then being dealt with. In the design of the tower it was the all-important force.

If we consider the final point of attachment of the cables to the towers we see that the horizontal forces intersecting at this point must always be in balance or motion will result. In other words, there is a definite position of equilibrium for this point that is compatible with a balance between the H's in the main- and side-span cables, and the horizontal compelling force P which represents the stiffness of the tower. With the cables in their maximum dead-load position this point of equilibrium is by hypothesis five inches shoreward from the position of the straight or unflexed tower. Let us suppose now that the saddle is on rollers. The point of intersection of the side- and main-span cables will then be free to take its balanced position independently of the resistance of the tower. When load is removed from the cables this balanced position will be displaced shoreward, and for the condition of the unloaded cables will become 18 inches from the vertical. While laying each strand in the saddle and while adjusting it to the proper sag, its unrestrained position in the sheaves is such that the point that is to be placed over the centre line of the tower is found to be a few feet down along its length in the side span. It is therefore necessary to pull a certain amount of each strand over into the main span when it is dropped into its correct position in the saddle. If the distance that a strand is pulled becomes too great, the strand will slip in the saddle. The full 18 inches in our case was too great, and therefore it was reduced to 7 inches by pulling the towers back 11 inches by means of independent guy ropes before erection was begun. This gave a strand friction of 9 per cent., and was found to be ample to prevent slipping.

The writer would like to dwell in passing upon the cable calcu-

lations made and intended to be used as a guide in closing the stiffening trusses. It was planned to coördinate the work of riveting the trusses and pouring the concrete slab on the roadway in such a manner that the two could proceed together. The trusses were fabricated and assembled in the shops in a position corresponding to their maximum dead-load position in the bridge, at which time they are assumed unstressed. Therefore before the chord splices or diagonal connections were made the cables had to be in, or very near, their maximum dead-load position. As about only two-thirds of the full load was on the structure at this time, the cables were high. However, by balancing loads on the three spans it would be possible to bring the cables down to their proper positions and close each span. The problem was one of assuming the normal sag and temperature for the span to be closed, and then finding for various uniform loads in the closing span the balancing loads in the other spans. From this simple information load-closing curves were plotted for different temperatures. These curves gave for a load on any span the load on the other spans necessary to hold the former one in its closing position.

An interesting and detailed analysis was made of suspender lengths. It was desirable to hang the stiffening trusses very close to their elevation as designed. In suspension bridges this is generally done by adjusting the suspenders, as the need may require, after the trusses are hung. Because there was no facile means of adjusting the length of these ropes after they were in position and after the trusses were hung from them, an attempt was made to establish their correct length from a comprehensive collection of field data. Numerous tests of the wire rope were made and analyzed, and a representative stretch factor was obtained with which to compute the elongation of the suspenders which would occur when the dead load of the structure came upon them. The length of the suspenders to be cut was calculated by using these stretches and by basing all figures upon theoretical elevations for cable and truss profiles. These results were sent to the field, the ropes were laid out, measured off, cut and socketed, and then remeasured. The variations involved in these field operations were thereupon transmitted back to the office. Here, in the meantime, data worked up from a field survey of the completed cables gave us the predicted elevations at every panel

point along the centre line of the cable in its final dead-load position; periodical reports from the shops where the stiffening trusses were being assembled also furnished us with the data from which we established the actual profile of the trusses in their fabricated position. These data even included the small shop errors in elevation that were made in assembling the suspender connection angles to the trusses. Finally, the variations in setting the cables and fabricating the trusses, combined with those made in actual cutting and socketing the ropes, were translated into terms of a correction in length for each and every suspender—596 in number. This length correction was physically made in the field by shimming between the suspender sockets and bearing angles on the trusses. The almost contemporaneous progress of the work of truss fabrication, suspender cutting, and even truss erection made coöperation very essential in carrying out this program.

There remains one phase of cable calculations not yet mentioned—the analytical computation of field check measurements. These measurements were taken in order (1) to determine the actual stiffness of the main towers in resisting horizontal forces applied at their top, (2) to establish the length of the uncompacted and compacted cables as they were placed, and (3) to predict future tower positions and cable elevations for the maximum dead-load condition.

At the time of pulling back the tower columns in preparation for cable erection, it was found convenient to check the tower stiffness. The field measurements consisted of observations of the sags in the guy-ropes used in pulling the towers back, the sags in the foot-bridge cables, and the temperature. The calculation required the finding of the net horizontal force at the top of the tower, and the portion of the reported deflection due to this force (the vertical load is responsible for some of the deflection). The results showed an average stiffness of 19,000 pounds per inch of deflection compared with the theoretical amount of 20,800 pounds per inch. This is a difference of 9 per cent. When the wide, possible variations in load and temperature combined with the difficulties attending the physical measurements are considered, this might be classed a very close check.

When the cable had been completely strung, and while it was still in its hexagonal, uncompacted state there was an opportunity offered for checking the actual lengths of wire in the

individual spans. Field measurements included observations of sag (obtained by surveying elevations) tower deflection and temperature. The computations consisted of finding the catenary curve fitting these conditions. A parallel set of observations and calculations were made after the cable had been compacted to its final circular shape. The results of both of these checks are shown in Table I.

TABLE I.
Cable Lengths from Field Observations.

Date of Observation.	Philadelphia Side Span.	Centre Span.	Camden Side Span.
	Theoretical Lengths.		
	780.60 feet	1791.60 feet	780.60 feet
	North Cable—Uncompacted.		
December 23, 1924	780.63 feet	1791.60 feet	780.60 feet
December 30, 1924	780.61 feet	1791.75 feet	780.56 feet
	North Cable—Compacted.		
February 6, 1925	.....	1791.76 feet	.....
February 25, 1925	780.64 feet	.....	.....
March 5, 1925	.....	.....	780.60 feet
	South Cable—Uncompacted.		
December 23, 1924	780.58 feet	1791.68 feet	780.63 feet
December 30, 1924	780.57 feet	1791.73 feet	780.61 feet
	South Cable—Compacted.		
February 6, 1925	.....	1791.77 feet	.....
February 25, 1925	780.61 feet	.....	.....
March 5, 1925	.....	.....	780.60 feet

With these check analyses on the unloaded cable for a basis, a prediction of final tower positions and cable elevations for maximum dead load was made. The actual observed catenary cable lengths were increased by the theoretical stretches occurring between the two conditions of loading. This gave the probable length of parabolic cables under full dead load. It only remained to find the horizontal spans required to balance the H's in the three cables. Table II shows the results obtained. Scanning this table we might interpret it as meaning that, from the field data on hand at the time, we would expect the cables for the completed bridge to hang at about the right elevation in the side spans, and about 1½ or 2 inches high in the centre span; and that we would expect the towers to be a fractional part of an inch shoreward from their designed position except in one case, the north Philadel-

phia tower leg, which would be less than an inch riverward from this position. All of this would be predicated upon the assumption that the total stretches and changes in load between the two conditions of unloaded and loaded cables would be as theoretically computed, and that the distribution of dead load would be uniform.

We have seen how important a part cable calculations play in the design of the towers, stiffening trusses and cables, and how

TABLE II.

Predicted Cable Elevations and Tower Deflections for Maximum Dead-load Conditions from Field Observations.

	Cable Elevations.		
	Centre of Philadelphia Side Span.	Centre of Main Span.	Centre of Camden Side Span.
Theoretical elevations	+219.80 feet	+176.80 feet	+219.80 feet
From observations: North cable	219.78 feet	176.93 feet	219.79 feet
From observations: South cable	219.80 feet	176.97 feet	219.80 feet

	Tower Deflections.	
	Philadelphia Tower.	Camden Tower.
Theoretical tower deflection	−5 inches	−5 inches
From observations: North column	−4⅝ inches	−5⅛ inches
From observations: South column	−5⅛ inches	−5⅛ inches

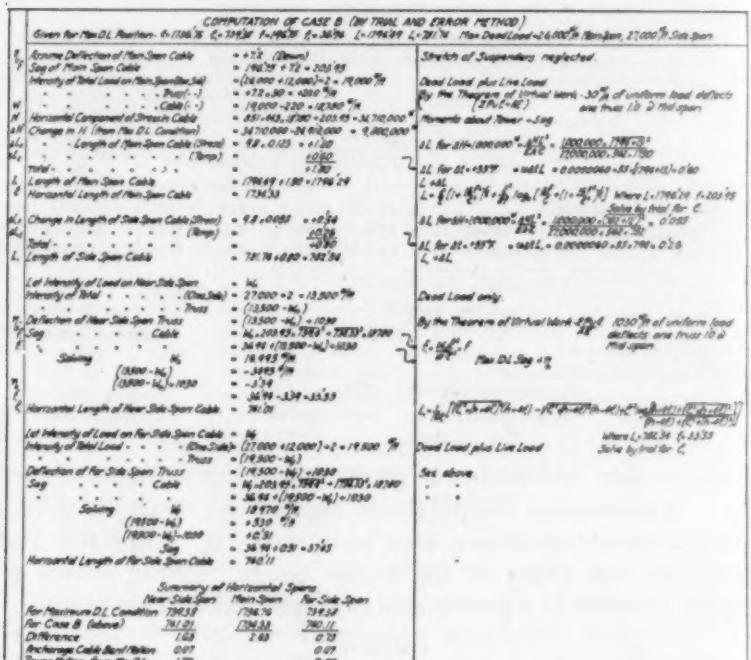
necessary they become in the planning and carrying out of erection. Mathematical complications, myriads of figures, and days and months of wearisome labor were involved. Altogether they form only one phase of the bridge design. Other phases are equally essential to a strong and economic and esthetic bridge.

APPENDIX.

Fig. 2 is a representative set of computations for determining the tower motions and forces used in the stress analysis of the main towers. This is the trial and error method explained on page 716. It was checked for each case of loading by an independent method after the cable stress and deflection in the adjacent side span had been determined by the deflection theory. On the right of the computations there appear explanatory notes and formulæ. The entire work is complete in itself except for the determination of the truss stiffnesses. As explained in the notes this involved an application of the principle of virtual work to the trusses. The trusses, constant in depth, have a variable

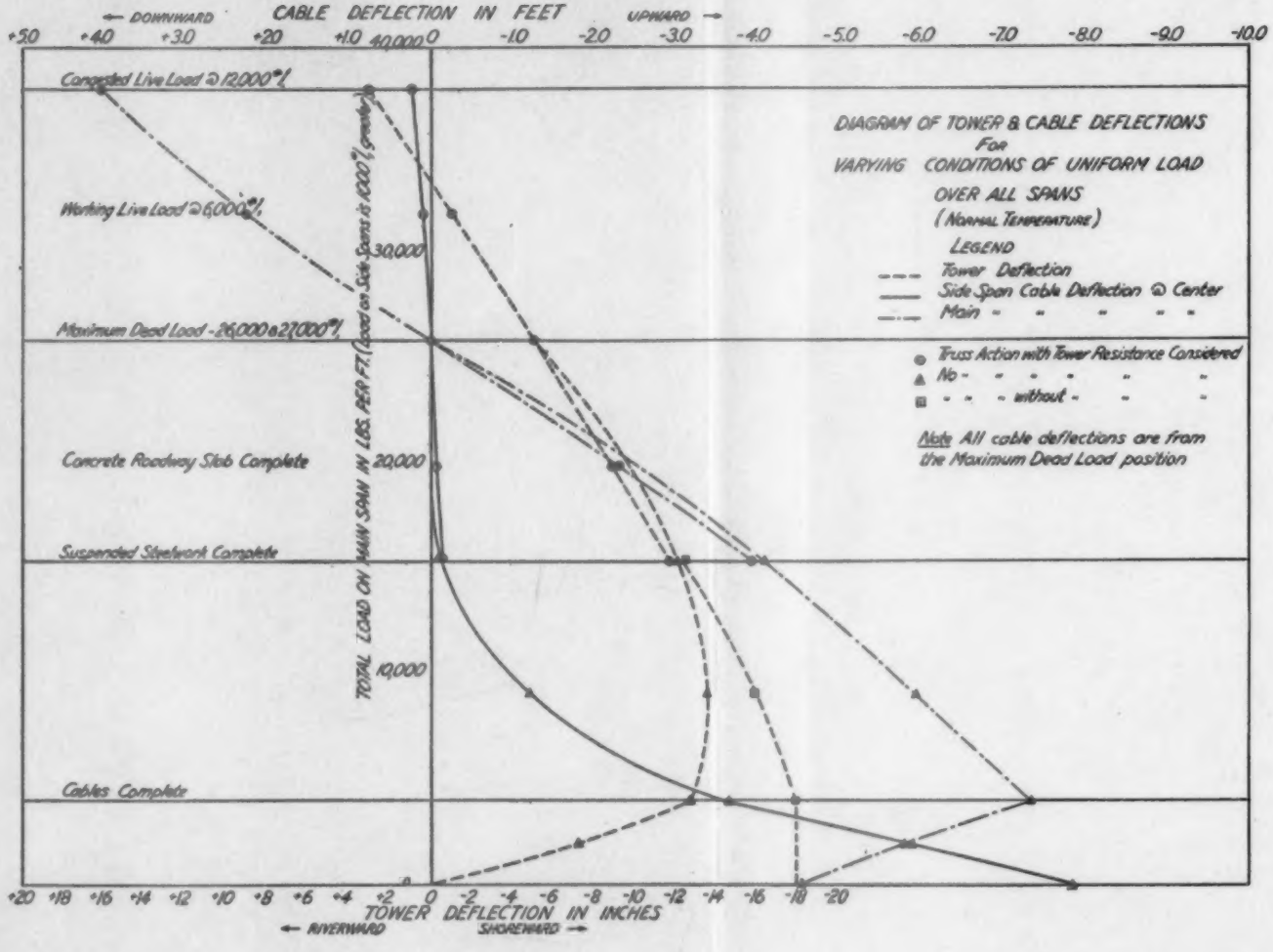
moment of inertia because of the proportioning of the chords and diagonals for the varying stresses. Moreover, the splice and gusset material is heavy on account of the use of the high-strength nickel and silicon steels in the chords and diagonals, respectively, and the reversal of stress in the truss members. Therefore the problem was to determine the moment of inertia of an equivalent

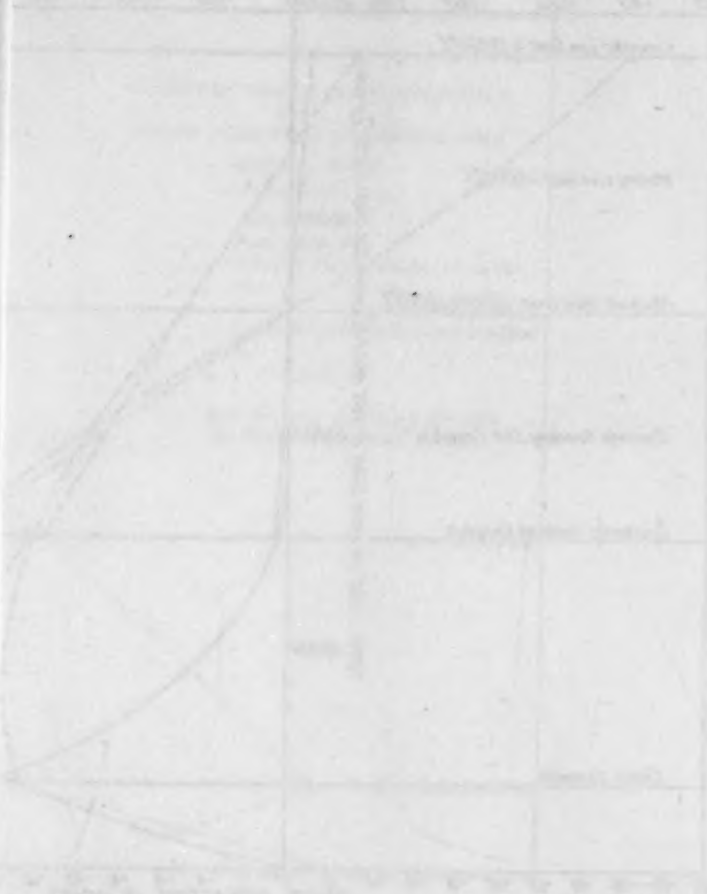
FIG. 2.


COMPUTATION OF CASE B (BY TRIAL AND ERROR METHOD)
Stretch of Suspenders neglected.
Dead Load only.
Dead Load plus Live Load
Moments about Tower = Sag.
Dead Load plus Live Load
See above
Summary of Horizontal Spans
Near Side Span
Main Span
Far Side Span
For Maximum D.L Condition 739.58 734.76 739.58
For Case B (above) 741.01 734.33 740.11
Difference 1.43 2.43 0.73
Anchorage Cable Band Motion 0.07 0.07
Tower Motion-Pure Max D.L 1.78 0.80
Decrease in Hor Length of Main Span Cable 2.43
Increase " " " Side " Cables 2.50
Error 0.07

truss or beam having the same length as the actual truss but a uniform moment of inertia such that the deflection of this hypothetical truss or beam would be the same as that of the actual truss for any conveniently assumed uniform load. $\Sigma \frac{Pul}{AE}$ was figured with refinements for both the side- and the main-span trusses. In these computations there were included the effects of shear deformation and the stiffening effect of the splices and gusset plates. In order to compare and simulate actual loading

FIG. 3.


← DOWNWARD
CABLE DEFLECTION IN FEET
UPWARD →
+5.0 +4.0 +3.0 +2.0 +1.0 0 -1.0 -2.0 -3.0 -4.0 -5.0 -6.0 -7.0 -8.0 -9.0 -10.0
Congested Live Load @ 12,000#/
Working Live Load @ 6,000#/
Maximum Dead Load - 26,000 & 27,000#/
Concrete Roadway Slab Complete
Suspended Steelwork Complete
Cables Complete
40,000
30,000
20,000
10,000
TOTAL LOAD ON MAIN SPAN IN LBS. PER FT. (Load on Side Spans is 1000#/ greater)
DIAGRAM OF TOWER & CABLE DEFLECTIONS
FOR
VARYING CONDITIONS OF UNIFORM LOAD
OVER ALL SPANS
(NORMAL TEMPERATURE)
LEGEND
Tower Deflection
Side Span Cable Deflection @ Center
Main " " " " "
Truss Action with Tower Resistance Considered
No " " " " "
" " " without " " "
Note All cable deflections are from the Maximum Dead Load position
+20 +18 +16 +14 +12 +10 +8 +6 +4 +2 0 -2 -4 -6 -8 -10 -12 -14 -16 -18 -20
TOWER DEFLECTION IN INCHES
← RIVERWARD
SHOREWARD →



conditions, figures of stiffness were made for the following conditions of loading on the main span: (1) Uniform load over the entire span, deflection at mid-span; (2) uniform load over one-half the span—deflection at the one-quarter point; (3) uniformly varying load, from zero at mid-span to a maximum at the ends of the span—deflection at mid-span; (4) two equal moments applied at each end of the span—deflection at mid-span. The results showed the following equivalent moments of inertia:

Case 1	84,900 in.^{2} – ft.^{2}
Case 2	83,500 in.^{2} – ft.^{2}
Case 3	83,900 in.^{2} – ft.^{2}
Case 4	79,900 in.^{2} – ft.^{2}

The value 84,400 in.^{2} – ft.^{2} was used in all final stress calculations.

The side-span truss stiffness was computed on the basis of a uniform load over the entire span—deflection at mid-span. This gave an equivalent moment of inertia of 112,500 in.^{2} – ft.^{2} which was used in all final computations of stresses.

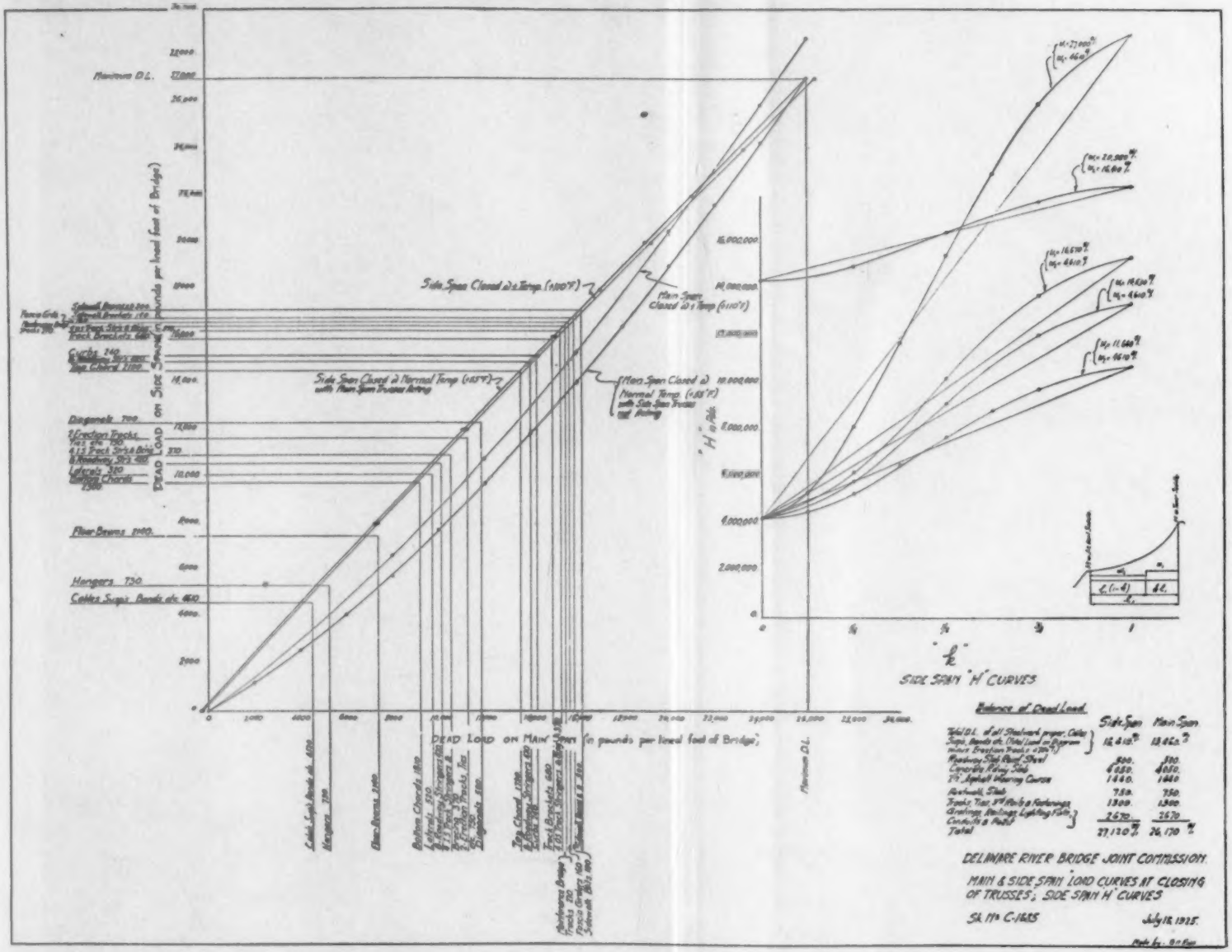
Fig. 3 shows the computed theoretical tower and cable deflections as the uniform load on the bridge increases from the time when one wire is stretched to the time when the full congested live load is reached. Below maximum dead load these figures represent in the main hypothetical conditions, and not conditions actually attendant upon field erection. They show the deflections which we would expect during construction were there no erection equipment such as foot-bridges, storm cables, and guy-ropes, etc., up at the time. They are therefore generally of academic interest only, but they tell us some interesting things and enable us to speak in generalities. For example, it shows us that when the cables were completed each tower would be deflected 12.6 inches shoreward and that the main-span cable would be 7.3 feet and the side-span cable 3.6 feet above its final maximum dead-load position, if there were no foot-bridges or other cable erection equipment up at that time. Following up the diagram with the increase in load until we reach the point when the steelwork was erected complete, we see the changes that occur during the different stages in erection of the stiffening trusses. Because of variations made in the loadings accompanying the several passes of the traveller in erecting the trusses, the deflections shown on the diagram only roughly approximate those attained in the field.

As we scan the chart and proceed upward from the suspended steelwork complete stage to the condition of maximum dead load, we note several things: That the side-span cable has stiffened perceptibly and that from now on a uniform increase of load over the entire bridge affects the deflection of these spans but very little; and that at maximum dead load the cable deflections are zero in both spans and that each tower is deflected five inches shoreward. It will be recalled that this condition of maximum dead load is our reference condition, and these deflections are therefore those fixed at the outset of design. For economy in combined stress distribution the towers were designed to be deflected five inches shoreward for maximum dead-load conditions. The double line for all deflections during this period represents the effect of the trusses in dampening cable deflections. Until the truss members are riveted, or at least well pinned, and until the end reaction pins are placed there can be no load taken by the trusses and they cannot therefore affect the cable deflections by virtue of their stiffness. In the field the process of riveting the trusses and the time at which the reaction pins were placed did not correspond to conditions represented by either of these double lines, for one line assumes that the trusses were completely riveted and that the pins were placed just when the erection of the steelwork was completed, while the other line assumed that these two operations were not performed until the maximum dead load was all placed. The actual field operations were performed over a period of time between these two limits of loading.

Above maximum dead load the diagram gives us the true expected deflections for conditions of full live load. As noted, all deflections are for normal temperature.

Fig. 4 is that of the load-closure curves referred to on page 729. These curves are plotted with intensity of dead load on the main span along the horizontal axis and intensity of dead load on the side spans along the vertical axis. Four curves are shown—two for the main span closed at highest and normal temperature, respectively, and two for the side spans closed at highest and normal temperatures, respectively. When any span is in its maximum dead-load position, that is, when the trusses are not deflected from their no-stress position, that span is said to be closed. During the riveting of the trusses and the pouring of the slab the temperature was well above the normal of 55° F., and

FIG. 4.


Maximum D.L.
Side Span Closed @ 0° Temp. (110°F)
Main Span Closed @ 0° Temp (110°F)
Side Span Closed @ Normal Temp (+55°F) with Main Span Trusses Acting
(Main Span Closed @ Normal Temp (+55°F) with Side Span Trusses not Acting
DEAD LOAD ON SIDE SPANS (in pounds per lineal foot of Bridge)
DEAD LOAD ON MAIN SPANS (in pounds per lineal foot of Bridge)
Floor Beams 2480
Hangers 730
Cables Susps. Bands etc. 460
Diagonals 700
Curbs 240
Lateals 320
Bottom Chords 1810
Top Chord 1780
Maximum D.L.
"H" in Kips
SIDE SPAN "H" CURVES
Balance of Dead Load.
Side Span
Main Span
DELAWARE RIVER BRIDGE JOINT COMMISSION
MAIN & SIDE SPAN LOAD CURVES AT CLOSING OF TRUSSES; SIDE SPAN H' CURVES
July 12, 1915.

therefore no curve for lowest temperature appears. Although the erection loads, that is, the slab sections as they were poured, were not strictly uniform loads over the entire span, they were generally fairly well distributed and consequently the assumption of uniform load distribution was made. It was anticipated in making these curves that the main span would be closed first and that all of the end reaction pins would be placed before the side spans would be closed. This was considered necessary because even when the slab in the main span was completed, and with the highest temperature, and with no slab in the side spans, the main-span cable could not be brought down to its maximum dead-load position while the trusses in the side spans were acting to hold it back. On the other hand, the side-span cables could be easily brought down to their dead-load position while there was truss action in the main span.

CONSTRUCTION OF THE CABLES OF THE DELAWARE RIVER BRIDGE.*

BY

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INTRODUCTION.

THE subject of this paper, which I have the honor of presenting to-night, is the construction of the two main supporting cables of the Delaware River Bridge now nearing completion. To many of my hearers the subject is probably clouded with mystery and, to a degree, awe-inspiring; while to many here it is already familiar; so that my task is somewhat complicated—to try to make clear to the former the operations that were necessary in suspending the 6500 tons, or 25,000 miles, of wire in their places in the 30-inch-diameter cables of this wonderful bridge, and to keep the interest of the other members of this distinguished audience and perhaps clarify some unfamiliar points of the work.

HISTORICAL REVIEW OF CABLE CONSTRUCTION METHODS.

There are three different methods of building suspension-bridge cables composed of high-tension steel wire, as follows:

First, the construction of cables composed of numerous wire

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ropes or wire-rope strands, laid parallel with one another, and bound or clamped together into a group forming a cable.

Second, the construction of cables made of parallel wires laid straight, side by side, by first laying the wires in groups of a given number of wires each, banded together into a solid cylindrical member known as a strand, along the shore or some other convenient place and then hoisting these strands to their correct place in the cables.

Third, the construction of cables made of parallel wires, by the "spinning-in-the-air process," the process whereby the wires are laid in the cables in their final position in the completed bridge, so to speak. This is the method on which I have been assigned to speak and which was used in the construction of the Delaware River Bridge cables and in all important suspension-bridge cables composed of wires laid straight and parallel since the adoption of this method by its inventor, the late distinguished engineer, John A. Roebling. This method was used by him for the first time, so far as the speaker knows, on the construction of the four ten-inch-diameter cables of the Niagara Railway Suspension Bridge over the Niagara Gorge at what is still known as Suspension Bridge, N. Y., about half a mile above the whirlpool rapids. The construction of this combined highway and railway international bridge connecting the New York Central and the Great Western Railway of Canada, afterward known as the Grand Trunk Railway, was commenced in 1850 and completed in 1854. It is the writer's understanding that the first physical connection across the gorge was made by Mr. Roebling by means of a kite flown from one side to the other; this is only mentioned as a matter of interest to those knowing the turbulent and impassable nature of the Niagara River waters at this point and to illustrate the resourcefulness often required of engineers to overcome difficulties.

Before returning to the strict subject of this paper, it may be of interest to note that there had been built at this time one longer-span bridge with parallel wire cables, the Ellet suspension bridge at Wheeling, W. Va., built by Colonel Ellet in 1848, the main cables of which are still in use and carrying a large traffic of motor vehicles, exclusive of heavy motor trucks. The cables of this bridge were each built of three strands laid up on shore and then hauled into their final position in the bridge, as previously

mentioned as the second method of constructing wire cables. This method has been employed since on all wire-cable suspension bridges over the Ohio River, except the Cincinnati-Covington Suspension Bridge completed by John A. Roebling in 1867, following the construction of the Niagara Railway Suspension Bridge and on which he again used the " spinning-in-the-air " method.

The method of the " spinning-in-the-air " cable construction has been gradually developed into greater speed of construction since the construction of Brooklyn Bridge to the present time, despite the fact that but a comparatively small number of long-span suspension bridges have been built. This has been due mainly to improvements in the type of foot-walks or temporary suspension bridges, improvements in machinery, the introduction of electric motor-power, and the improvement in tools of construction. To illustrate and make clear what this development amounts to, the speaker will now proceed to the description of the work on the Delaware River Bridge cable construction.

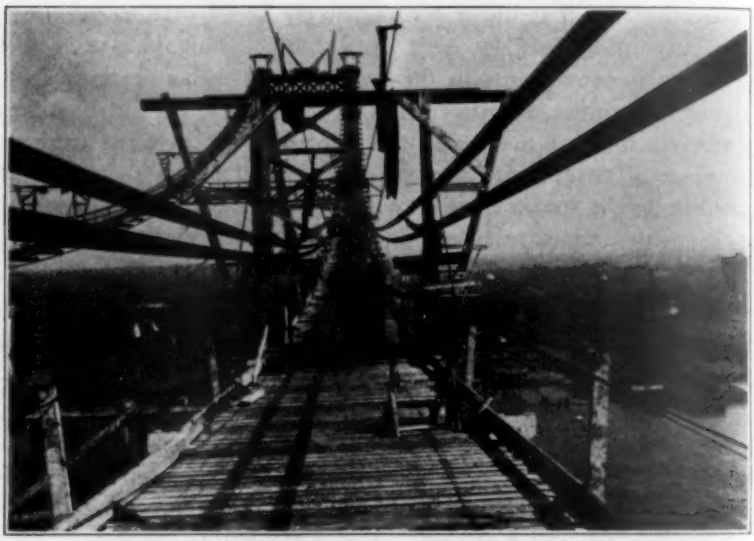
OUTLINE OF THE PROBLEM.

Starting with the specifications and plans for cable construction as issued by the Commission Engineers, the problem laid before the constructor was the completion in place of two main bridge cables, each composed of 18,666 No. 6 galvanized steel wires, divided into sixty-one strands of 306 wires each, and having a total length of approximately 3548 feet between their end-connections to the anchor-chain bars in the masonry anchorages in Philadelphia and Camden. The elevation of mid-span sags for both the main- and side-span portions of the unloaded cables had been prescribed by the Engineers, also the positions of the tops of the two main towers under the weight and balance of cables by which the elastic towers were bent shoreward. With this information and the known desired elevations of the cables at mid-spans when the total assumed dead load of superstructure should be placed with tower tops at a certain desired bent position, and other information in regard to elevation and position of end-connecting anchor chains and inclined bents supporting cables where they bend downward at face of anchorages, the constructor had to start his work.

First, he had to compute the lengths of the cables in their final position with all dead load of structure in place, then compute

the amount of elongation of the cables due to all the suspended dead load and obtain the unloaded length and sag elevations of the cables for the assumed normal temperature. Up to this point he had been dealing with parabolic curves as the Commission Engineers had decided to assume the entire dead load as evenly distributed per horizontal foot of span instead of a variable load. Now his problem involved the more complicated mathematical catenary curve requiring much more arduous work, and its first

FIG. 1.



Foot-bridges—Looking east on foot-walk from middle of centre span.

application was in the determination of the amount of unbalance of cable forces exerted at tower tops in the supporting saddles from the unloaded cable in place and of span lengths already determined, assuming the towers in their unrestrained vertical position.

BALANCING OF THE STRANDS.

As it was the wish of Mr. Ralph Modjeski, Chief Engineer of the Board of Engineers, to let the strands of cable produce the resultant bending of main towers shoreward due to the inequality of horizontal forces as the strands were laid in the saddles, every possible way was considered and investigated for accomplishing the desired end. The unbalanced forces between land- and main-

span cables at the main-tower saddles were not so great but that the frictional resistance in the saddles would hold the individual strands, but there were other very important details of the construction to be considered, *viz.*, the strands of the cables during their spinning.

It has been the custom on all recent suspension-bridge wire-cable constructions, and it is desirable, to have the temporary foot-bridges laid parallel to and just below the main cables with

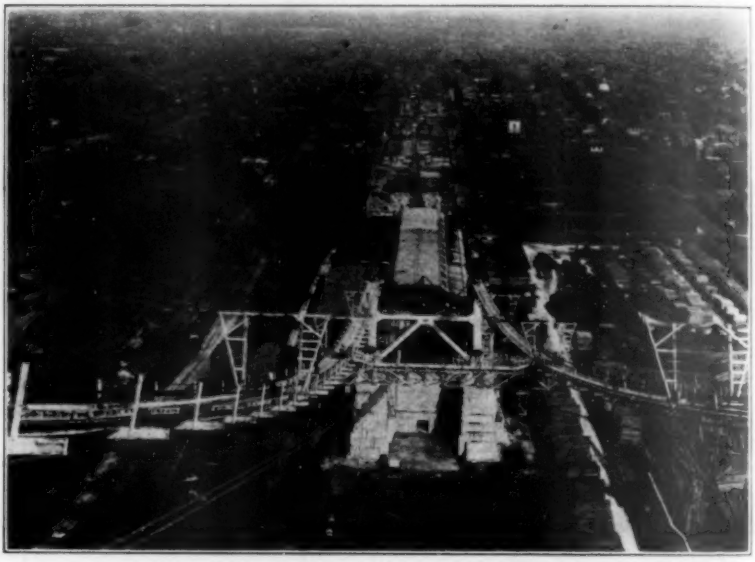
FIG. 2.

Camden—General view of bridge looking northwest from Delaware Avenue.

sufficient clearance for the operation of the cable-squeezers and wrapping-machines. This was the plan adopted at Delaware River. But the strands being constructed alongside the main cables are supported in sheaves over the main-tower and anchor-bent saddles, which sheaves are free to turn on their trunnions, thus allowing the strands to hang in their balanced position with equal catenary parameters in the spans corresponding to equality of strand stress and consequent balance in supporting sheaves at main saddles. This balanced position of strands decreased the sag so much in the main span and increased it so in the land spans for every possible position of supporting sheaves and position of

strand-shoe on the "leg" at anchorages that it was found impossible to rely upon the individual strands to bend the towers; the use of "pull-back" ropes was therefore recommended and the approval of their use was given by the Board of Engineers. It was determined by computation that the tower tops would require to be bent shoreward at least eleven inches when the temporary foot-bridges were completed with storm cables, etc., in place before stringing of strands could proceed. This was barely

FIG. 3.



Foot-bridges—Side span looking east from tower B.

enough, it was found, to hold the strands clear of the foot-walks in the land spans and not have the strands at too great height above the foot-walks in the main span, as is well illustrated in Fig. 1 for the main span and Figs. 2 and 3 for the land spans.

THE TEMPORARY FOOT-BRIDGES.

The temporary foot-bridges for this work were similar to those used in the construction of the Manhattan and the Bear Mountain Bridge cables, and consisted of a timber walk 10 feet wide supported by two cables for each walk, each cable composed of three 2¼-inch-diameter galvanized steel ropes which were later to be used as suspenders for the main bridge superstructure.

The timber walk consisted of 8-inch × 12-inch × 16-foot Oregon fir floor-beams, spaced the same horizontal distance centre to centre as the panel lengths of main structure. Upon these floor-beams were laid four lines of 3-inch × 10-inch Oregon fir longitudinal stringers. The flooring consisted of 6-inch × 1¼-inch × 10-foot planking laid with clear spaces of 1¼ inches between planks. The speaker originally planned to provide seven transverse cross-bridges or walks connecting the two main foot-walks in the main span and three in each land span with diagonal ⅞-inch-diameter rope-tie lateral bracing between. A question as to sufficient lateral rigidity being raised, and knowing too well how impossible it is to make an inflexible foot-walk, he decided to leave out four of the main-span cross-walks and two of the cross-walks in each side span, and to dispense with the inefficient diagonal rope bracing entirely. The resulting walks proved very satisfactory for the work for which they were to serve and sufficiently rigid for all purposes, except possibly to convey across an occasional curious but nervous visitor.

STORM CABLES.

To check the vertical oscillations of the foot-walks, an inverted parabolic wind cable, consisting of a single 2¼-inch-diameter galvanized steel wire rope, was stretched under each main foot-walk cable in both main- and land-span cables and connected to the foot-walk cable at intervals of about 100 feet, or five panels of foot-walk, by ¾-inch-diameter vertical rope suspenders. These foot-bridge storm cables were connected to temporary fastenings near the base of the main-tower columns and to the main stiffening-truss rocker-arm pin-connection on the inclined cable bents at the anchorages. These storm cables were provided with adjustable end-connections and were stretched up to a panel sag giving an initial stress of approximately 10,000 pounds in the rope.

OTHER PROVISIONS FOR THE CABLE SPINNING.

Five timber gallows-frames on each foot-walk of the main span and two in each land span were erected for supporting the travelling rope supporting sheaves, dividing the rope spans into approximately 250-foot lengths. This distance has been found by experience to be about the maximum efficient span. On the top of each main tower were placed two ten-ton stiff-leg derricks, one to command each main-cable saddle, and a suitable and safe steel-

floor working platform. The cast-iron saddles provided by the cable constructor were supported on temporary steel brackets attached to the main-tower columns on the outside and to the top

FIG. 4.

Cable bent—Saddles for foot-bridge ropes, Camden.

transverse portal bracing on the inside. At the inclined cable bents on the anchorages, the foot-walk cable saddles were also supported on steel bracket-framing attached to both the inside and outside faces of the columns of these bents, as shown in Fig. 4. These

brackets also supported the steel gallows-frames at both the main towers and inclined cable bents as illustrated in Fig. 5. These steel gallows-frames carried the supporting sheaves for the wire-spinning hauling ropes, also the carriages supporting the hydraulic pulling-jacks used in lifting strands from the supporting sheaves after their stringing was completed and placing them in their places in the main-cable saddles. One 10-ton stiff-leg derrick was erected on each anchorage at the rear to reach the end of each

FIG. 5.

Foot-bridges—Looking northeast from Front and Race Streets.

main-cable anchor chain. Reel-stands, each to accommodate three reels of approximately 80,000 feet of main-cable wire, were placed along each side of each anchor chain at the rear of the anchorages, and timber working platforms were erected on the anchorages at the level of the end of the chains over a large area of the anchorage on each side. Inclined steel bents about 70 feet high were placed under each main cable approximately normal to its direction from the saddles on the inclined bents to the ends of the anchor chains, to support the strands of the cables during construction in their final position at the point where they begin to splay out to their attachments to the anchor chains.

THE DRIVING MACHINERY.

The machinery for operating the wire-spinning was located on the Philadelphia anchorage and consisted of a heavy-duty, mill-type, 50-h.p., back-geared motor directly connected to mitre gears on a vertical shaft which carried the main hauling-rope traction-wheel, 6 feet in diameter. This machinery with the necessary idler and deflection sheaves was located on the centre line of each main cable at the rear of the anchorage. An endless loop of ¾-inch plow-steel hauling rope for the cable-spinning work was attached to this driving machinery in the usual way and extended along the spans from anchorage to anchorage over the centre line of each cable. At the Camden anchorage the loop or endless-rope passed around an idler sheave attached to a timber frame anchored to masonry which had provision for taking up a considerable amount of slack as the hauling ropes stretched out in use.

CONSTRUCTION OF THE FOOT-BRIDGES.

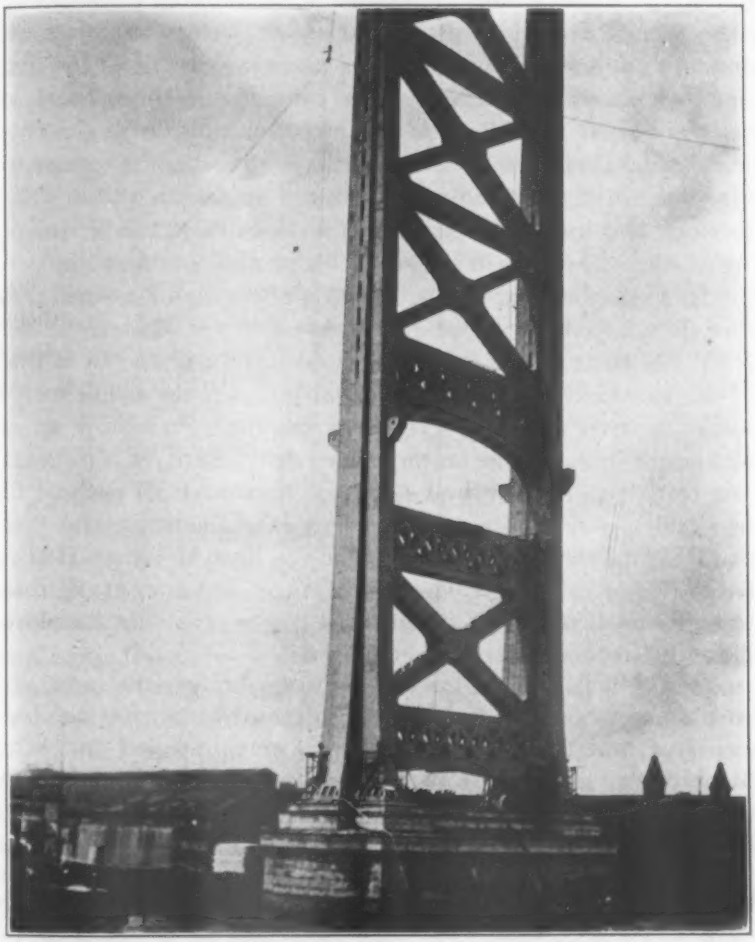
The construction of the foot-bridges was done in the customary way with some exceptions. The tower and anchorage derricks were set in place as soon as the main tower and the anchorage construction was in readiness, and the timber working platforms on the anchorages, the steel working platforms on the tops of the towers, and the steel bracket-framing to support the foot-bridge cable saddles at the main towers and inclined cable bents, were erected. An elevator was installed at each tower, providing for the conveyance of a limited number of men between base and top of tower. At the same time this work was under way, the timber for foot-walks, cross-bridges, hauling-rope-sheave gallows-frames, reel-stands, etc., were being framed. The foot-walk was assembled in complete panel lengths, marked and stored in regular order of erection to be ready to place upon completion of the foot-bridge cables.

ERECTING THE FOOT-BRIDGE CABLES.

The 2¼-inch-diameter foot-bridge cable ropes, each about 3590 feet long, were delivered on reels ready to be mounted in the reel-stands on the flat boat or scow used for their transportation across the Delaware River. The scow containing three reels of rope, mounted on trunnions in the reel-stands, was moored at the river side of the Philadelphia main tower; the ends of the ropes

were unreeled and pulled back to the Philadelphia anchorage, clear of the heavy traffic of Delaware Avenue, and the adjustable socketed end-connections were permanently secured to the foot-

FIG. 6.



Philadelphia tower. Hoisting first rope for foot-bridges.

bridge-cable anchor-bars previously provided in the main-anchorage contract. This scow was then towed across the river sidewise and, as the scow proceeded, the ropes were allowed to pay off the reels and sink to the bottom of the river. The scow was

then moored at the base of the Camden main tower, and the remaining rope on each reel was unreeled and laid in a long loop on the deck. The other ends of the ropes were then taken, one at a time, back to the Camden anchorage and a temporary end-connection made at the corresponding foot-bridge-cable anchor chain there. The bight of these ropes lying over the main-tower masonry on the outside of the tower base on each side of the river was then placed in a suitable sheave connected to the fall-lines of the main-tower derricks. At a convenient time, with the river patrolled and kept free of traffic at the bridge site, the ropes were lifted out of the water, one at a time, by means of the tower derricks, and the bights placed in the foot-bridge saddles on the main towers as shown in Fig. 6. This method was first employed by the speaker in placing the cable-rope strands of a small 400-foot span suspension bridge over Grasse River at Massena Center, N. Y., in 1910, where the power used was furnished by a team of horses; there the bight of rope was placed in the saddle on one side of the river and then, ferrying across, the bight of rope-strand was placed in the saddle on the other side of the river, this operation being repeated until all seven of the strands of each of the two cables were in place. It was so easy and successful there that the same method was employed at the Bear Mountain-Hudson River Bridge in the early summer of 1924, and later at the Delaware River Bridge, but steam horse-power was substituted for the animal horse-power first employed.

On the Williamsburgh and Manhattan bridges the same general method of getting the foot-bridge cables across had been employed, but the ends of the ropes were unreeled and taken directly over the tops of the towers and back to the anchorage fastenings on the Manhattan side of the river and, after towing across and sinking the ropes to the bottom of the river, the remainder was unreeled and laid in long loops on the deck of the scow and the ends were taken over sheaves on top of the Brooklyn tower. Hauling tackle was then connected to the ends of the ropes, one at a time, and they were pulled back to the anchorage connection by means of steam hoists on the Brooklyn anchorage. It was only a matter of five or six minutes after starting the engine when the ropes would be swinging in their approximate final position.

ADJUSTING THE FOOT-BRIDGE CABLES.

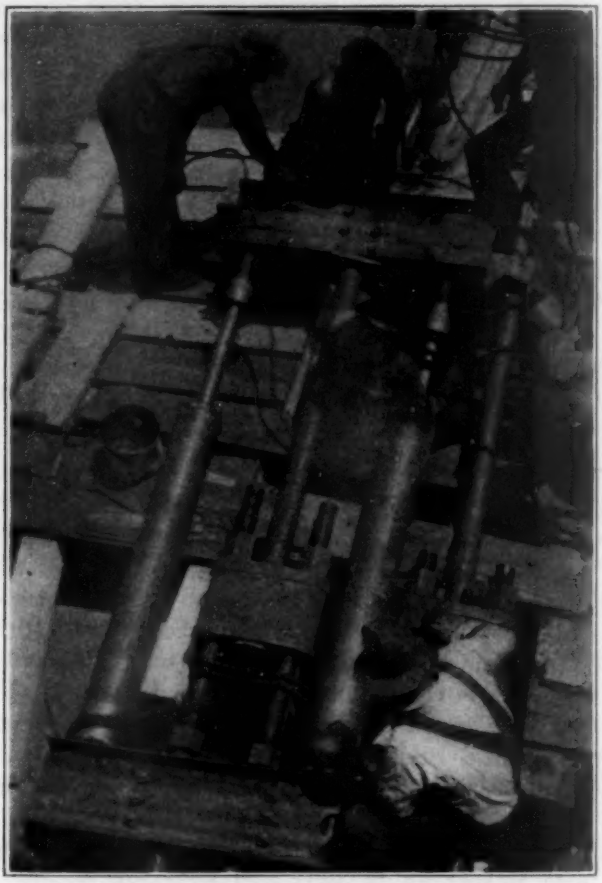
As soon as the first 2¼-inch rope was suspended in place over the Delaware River, its sag was measured in the main and side spans by level and transit and the length computed. This was repeated for one rope of each of the four foot-bridge cables. The lengths of these ropes were computed from the sag data obtained and then instructions were given, to the superintendent in charge of the work, for cutting off and socketing the ropes to the computed unloaded length required for the foot-bridge cables. The two remaining ropes forming each of the four foot-bridge cables were then hung to the same sag as the one already in place, and cut off and socketed. The ropes were then moved through their saddle bearings on the main towers until they were hanging at their theoretical unloaded mid-span sag in main and side spans, and U-clamps placed around each cable at the river face of the main-tower foot-bridge cable saddles to prevent possible slipping of the cables into the land span due to a considerable unbalance of rope-pull or, technically speaking, a difference in catenary parameters.

ADJUSTING THE "PULL-BACK" ROPES.

The four 2¼-inch-diameter pull-back ropes from the main-tower tops to the anchorage on each side of the river, previously mentioned, were then cut to their computed unloaded lengths, socketed and put in place. Some ingenuity had to be exercised in placing these ropes, due to the stretch cut out of their final length, amounting to nearly five feet. The ropes already attached to the temporary hitches on the top transverse portals of towers were pulled back, one at a time, by a double set of three-sheave block tackle until the ends could be caught by two of the 40-ton pulling-jacks to be used later in the construction of the main cables. In order to get lead enough for the hydraulic jacks to work, a length of double extra-heavy hydraulic pipe and a heavy square plate washer at its outer end were put on the end of the rope before socketing, as illustrated in Fig. 7. The inner end of the hydraulic pipe bore against the face of the rope socket. A yoke composed of two channel-beams straddled the rope and bore against the washer on the end of the pipe. The two 40-ton pulling-jacks had their plunger ends attached by pins to this yoke and the cylinder ends of the jacks were pin-

connected to similar yokes straddling the eye-bolts attaching the rope socket to the cast-steel bases of the inclined cable bents, the latter yokes being in bearing against extra nuts on the threaded shank of eye-bolts as near the base of the column as possible.

FIG. 7.



Hydraulic tackle for adjusting "pull-back" ropes.

It was then an easy matter to put the final strain of approximately 132,000 pounds on the rope and, when all four ropes were connected and stretched up to the same sag, to have the tower top bent shoreward to the desired deflection of 11 inches from plumb position.

ERECTING THE FOOT-BRIDGES, STORM CABLES, CROSS-BRIDGES, AND GALLOWS-FRAMES.

The work of laying the timber walks then proceeded in the main span from the tops of the towers towards the middle and,

FIG. 8.

Foot-bridges—Philadelphia side, looking east from beside anchorage.

in the land spans, from the tops of the towers towards the anchorages, as shown in Fig. 8. The panels of foot-walk already assembled with one floor-beam attached to each panel were then hoisted from the base of the tower to the top and placed on the foot-walk

in their order. Each panel beyond the first one placed was landed on top of the walk and skidded along same to its place until the walk was completed, a matter of about five and one-half days. The storm cables for the main and side spans were hoisted to the tops of the main towers, already cut to computed length and socketed. They were then laid along the foot-walks, one on each side; the points for attachment of suspenders were located by measurement and the suspenders attached to them. The other end of each suspender measured to figured length was laid over the side of the walk and attached to its point of suspension from the foot-walk cable. After the completion of this work, the 2¼-inch-diameter storm-cable ropes were lowered over the sides of the foot-bridge and their ends pulled out to the connections at the base of the tower and on the inclined bents at anchorages. They were then adjusted to the desired initial stress of about 10,000 pounds each. The three main-span cross-bridges, already assembled complete on a barge, were then hoisted to their respective places and attached to the underside of the foot-walks. The side-span cross-bridges were placed in a similar manner, but the work was more difficult to accomplish on account of the trouble in getting them past the "pull-back" guy-ropes already in place. The gallows-frames for supporting the travelling-sheave hauling ropes were then set up on the foot-walks, the ¾-inch-diameter hauling ropes were stretched in place over the spans, and all the small details in connection with the stringing machinery, too numerous to mention, were completed.

PLACING THE GUIDE-WIRES.

The next operation, one requiring careful computation and accurate work in the field, was the laying of guide-wires across the spans and their correct length determination. The guide-wires are made of the same size wire as the main-cable wire; as a matter of fact, the wire was taken from one or more of the reels of wire already placed on the anchorages. The end of a wire was pulled off the reel, the bight was thrown over the travelling sheave to be used in cable spinning and the sheave was run across the river, thus laying along the foot-walk two wires from which two guide-wires were made. As four strands of a cable were to be constructed at a time on each cable, four guide-wires were required for each of the two main cables under construction.

Whether this was done or not the speaker does not recall, but all four guide-wires for one cable could have been laid at one passage of the travelling sheaves across the span, two wires being hauled from the Camden side and two from the Philadelphia side.

ADJUSTING THE GUIDE-WIRES.

Preliminary to the description of the adjustment of the guide-wires, it will be well to explain their function. The guide-wires may be described as measuring rods or means used for getting the approximate lengths of the various strands of the cable at the beginning of their spinning. The final lengths of the strands are determined by their position in the cable saddles and on the anchor-chain connections at the ends, and by their accurately measured theoretical sags in the spans.

One guide-wire for each cable was laid in the strand-groove of the main tower and inclined anchorage-bent saddles, and its end was attached to the pin of the pair of eye-bars of the anchor chain at each anchorage for the selected one of the five bottom strands of the main cable it was intended to measure. It was then stretched up to a predetermined sag in the main span; the measurement of this sag was made by an engineers' level set at a convenient elevation in the Philadelphia tower to observe the target on an inverted level rod projected down through the foot-walk at the middle of the span. A graphical diagram of elevations of the centre line of the unloaded main cable at mid-span had been made from the computed elevations for various temperatures and bent positions of the tower tops. From this diagram, and with a standardized thermometer hanging nearby, and with the positions of the tower tops known from observations of plumb lines hanging inside of the tower columns, the engineers set the first guide-wire to its correct theoretical sag in the main span. A point on this guide-wire was marked on it and in the saddle-groove for observation, and the guide-wire was clamped at both ends of the saddle to prevent its slipping either way in the saddle. The guide-wire sag was determined in a similar manner in each land span, but the work was a little more complicated as the middle-point of span where the sag was to be measured had to be determined by use of an engineers' transit set on a transverse station point at some distance laterally from the axis of the bridge. On the Philadelphia side the point was on the roof of a pier shed some

distance north of the bridge. On the Camden side the station point had been established on a dock some distance south of the bridge. The sag elevation was read from an inverted level rod extended below the foot-walk floor by an engineers' level in the Philadelphia tower for that side span, and in the Camden tower for the Camden side span. A graphical diagram for the land-span cable sags similar to the main-span diagram was used with temperature and tower deflection observations considered as before. Thus the total length of the first guide-wire for each north and south main cable was determined and permanently marked on the wire. The three remaining guide-wires to be used in the construction of the strands were then laid one by one alongside of the guide-wire already hanging in its place and set at the same sag, and were thus measured in length between certain fixed points. A table of centre-line lengths of all of the sixty-one strands of each cable had been computed and checked so that now all that was necessary to change the guide-wire from one strand length to any other was to add or subtract its difference at each end where adjustment had been provided by means of a turn-buckle for the purpose.

STRAND-SHOES AND STRAND-LEGS.

Coming now to the actual cable-spinning operations, a brief description of the principal tools and appliances used in the work will be made as their introduction is mentioned herein. The strand-shoe around which the wires of a cable strand are laid at each anchor-chain connection has the approximate outline of a horse-shoe, hence the name. It is a steel casting having a groove of sufficient cross-sectional area to accommodate one-half the number of wires of the strand: In the case of the Delaware River Bridge cables, 153 No. 6 wires. The strand-shoe has an elongated pin-hole, to allow for the adjustment at each end necessary in obtaining the correct length of strand when regulated to its final position in the cable. In the case of the Delaware River Bridge, about 9¾-inch adjustment was provided at each shoe. The elongated pin-holes have a flat milled bearing at their back or anchorage end, and a bearing pin-block is provided with one face milled to the radius of the pin and the other face milled flat for bearing against the steel adjusting shim plates inserted between it and the flat bearing face of the shoe. During the stringing of the strand,

the shoe has its pin-axis turned through an angle of 90 degrees to a vertical position and what is termed the " strand-leg " supports it on the pin in the ends of the two eye-bars that finally connect

FIG. 9.

Cable strand on shoe at eye-bars.

it permanently to the anchor chain. The strand-leg serves two major purposes: To hold the shoe in a position so that the bights or loop of wire can be laid around it during the spinning, and to provide a location of the end of the strand in relation to the end-

pin of the anchor-bars such that a degree of control of sag elevation of the strand in the three spans can be obtained. In the Delaware River Bridge cable work, this location was approximately vertically above the eye-bar pin-centre as shown in Fig. 9. The strand-leg is a steel casting with a vertical web provided with a half-pin-hole to engage the end-pin of the anchor chain and projecting below the eye-bars just back of their heads far enough to have a slotted hole for the insertion of a steel key or bar of steel to take up the reaction of the completed strand pulling on its anchoring post or boss against which the shoe rests. This slotted hole is long enough to allow for all adjustments for alignment of strands from the shoe to the first supporting strand sheave at the main-cable saddle on top of the inclined cable bent at the front of the anchorage. The strand-leg has a horizontal platen on which the shoe lies during the spinning of the strand, and the strand-shoe is held against any possible kicking off the mooring post by a pin or bolt through the post just above the strand-shoe, as illustrated in Fig. 9.

STRINGING THE WIRES.

At the commencement of stringing of a set of strands, the guide-wires are first adjusted to the centre-line length of these strands by means of the turnbuckles provided at the ends, and are hooked to the eye-bolts in the strand-legs already set in place with the shoes attached. The guide-wires are then laid in the strand-supporting sheaves on the main-tower and inclined-bent main-cable saddles, and hang free in their balanced sags ready to measure the first wires of the strands when laid in place. This is well illustrated in Fig. 10, where the shoes of two strands are shown in place on the legs with the guide-wires attached, and around the nearer one of these shoes is seen the bight of wire thrown preliminary to stringing. The end of this wire is made fast at some convenient point a short distance away, probably in this case the temporary steel-bent supporting the cable strands at the splay point. Enough of the wire is then pulled off the reel from which the strand-wire is being strung and the bight or loop thus formed is thrown around the travelling sheave that is to haul it to the opposite anchorage fastening. Simultaneously the same operation is being conducted on the opposite side of the river for the corresponding strand which is being spun (for the same cable) by the travelling sheave attached to the other part of the travelling

rope and carrying a strand from a reel of wire on the other anchorage. As soon as the workmen adjusting the wires on the Camden anchorage were ready to start the stringing, the signal-bell button was pressed and, if all was ready at the Philadelphia anchorage, the motorman there started the hauling-rope sheaves across the span. As soon as the travelling sheave passed the saddle on the inclined bent, the standing or "dead" wire of the bight being hauled across the river was guided by the workmen

FIG. 10.

Strand-shoes in place for spinning—with guide-wires.

into the strand-supporting sheaves there, and the running wire was guided into small carrier sheaves that support the running wire in its passage across the span. These small sheaves were also located at intervals along the walks, in fact at every gallows-frame supporting the main travelling rope, and at these same points were also located suitable hooks to confine the standing or "dead" wire and prevent it from surging up and down to the discomfort and danger of workmen as the strain on the wire varied. When the travelling sheave passed the main tower, and the dead wire had been laid in the strand-supporting sheaves and the "live" wire guided into the small wire-supporting sheaves, the regulation of side-span "dead" wire was begun. The wire-regulating crew on the main tower took hold of the wire with the "come-alongs"

or wire clamps to which were attached small double-sheave tackle-blocks which one man manipulated along the main-span walk, pulling up or slacking the wire until it was adjusted to the same sag as the guide-wire, as determined by the wire regulator stationed at midway of the side span. The wire was then securely clamped to the guide-wire at the main tower and tied to it by means of short twine lashings at intervals of fifty feet or so along the span. A pencil mark was also made across the guide-wire and the wire just regulated at the saddle, to detect any possible slipping of wire. The signal was then passed to the main tower on the opposite side of the river that the "dead" or standing wire was then ready to be regulated in the main span and, as soon as the travelling sheave had passed that tower, the adjustment of the main-span wire by the wire-regulating force on that tower was begun. The regulator at mid-span signalled to the men at the tower to raise or lower the wire, pulling in or slacking on the tackle attached by the "come-alongs" to the "dead" wire until it was hanging parallel to, or with the same sag as, the guide-wire in the main span. The "dead" wire was then clamped to the guide-wire at the tower and lashed to it throughout the span at intervals of fifty feet or so by the twine mousings as previously mentioned. If the work progressed smoothly, by the time the travelling sheave reached the anchorage and the bight of wire hauled across by it had been removed and thrown around the shoe with the "dead" wire on the same side of the shoe as at the opposite anchorage, the "dead" wire was ready for regulating in that land span. This was done as soon as a bight of wire for the adjacent strand being strung from the reel on that side of the river had been thrown around the sheave and was started on its way across the span again in the opposite direction.

While the travelling sheave was going from that anchorage to the main tower, what had been the "live" or running wire on the other strand would be regulated in the land span and a signal passed to the opposite tower that the main-span "live" wire was ready for regulating. Then the same regulating force would begin the regulating of the "dead" wire of the bight of wire being hauled by the same sheave to the opposite side of the river. As soon as the men on the opposite tower regulated the "live" wire of the first strand and signalled to the crew at the anchorage that the land-span wire was ready for regulating, the tower crew

would begin the regulating of the "dead" wire, in the main span, of the other strand being strung from the Camden anchorage, if the travelling sheave hauling the bight for that strand had passed the Philadelphia main tower. If there was no unusual delay, the regulating of the "dead" wire above-mentioned would be completed by the time the travelling sheave had reached the Philadelphia anchorage, the bight of wire removed from the sheave and laid around the shoe and another bight for the opposite strand placed on the sheave ready for another trip across the span. The "dead" wire would be regulated in this land span and, immediately afterward, signal would be passed to the main-tower crew to take up the regulating of the "live" wire of this strand which had already been removed from the small running sheaves and placed in its proper position in the strand-supporting sheaves. Thus a round trip of one travelling sheave has been made, laying two wires in each of two strands on one side of the main cable; that is, it has strung two wires of one strand from a reel on the Philadelphia anchorage and returned with two wires from a reel on the Camden anchorage for the other strand on the same side of cable. At the same time, the travelling sheave on the opposite side of the cable, being operated by the other part of the travelling-rope loop, was similarly engaged on two more strands of the same cable. In this manner four strands of one cable were being spun at the same time. An entirely separate plant was performing the same operations on the other main cable, so that eight strands of the two main cables were being spun at the same time. This is clearly shown in Fig. 1, showing the first four strands of the south cable under construction. When a dozen wires had been regulated in place in a strand, the corresponding guide-wire was removed. This was speedily accomplished by lifting up the bight at each tower and laying the guide-wire along the outside of the walk.

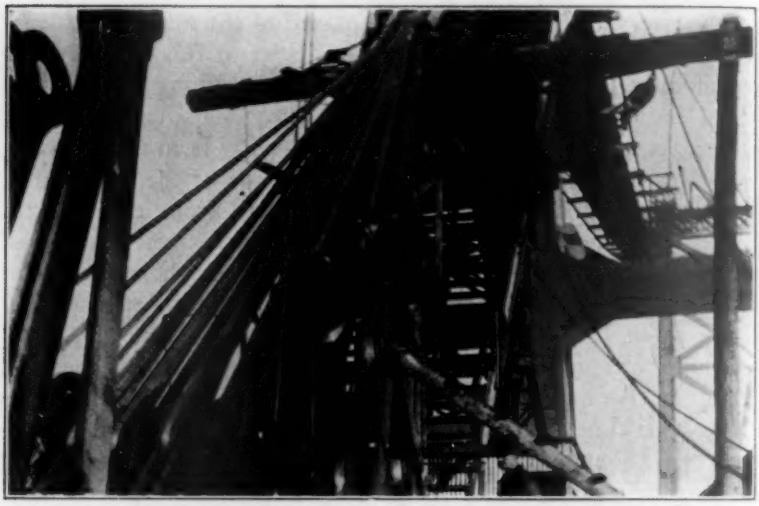
HANDLING THE COMPLETED STRANDS.

As soon as all the 306 wires of each of the four strands of a main cable had been strung in place, or all of the wires of any one of these strands were in place, and the two ends of wire forming the completed loop or skein had been brought together under the initial load stress, cut to correct length and spliced, the work of squeezing up the strand by the strand-tongs, placing the flat-wire seizings, and getting everything in readiness to

remove the strand from the temporary supports and fastenings and lay it in place in the cable was begun.

At the anchorage, 60-ton capacity hydraulic pulling-jacks were attached to one of the strands, the anchorage-end being connected to a special hook engaging a pin of the anchor chain. The piston-end of the jack was attached to a wire-rope pennant which, in turn, was pinned to what is termed a "bridle." The "bridle" is illustrated in Fig. 11 and is here shown in position on the end

FIG. 11.



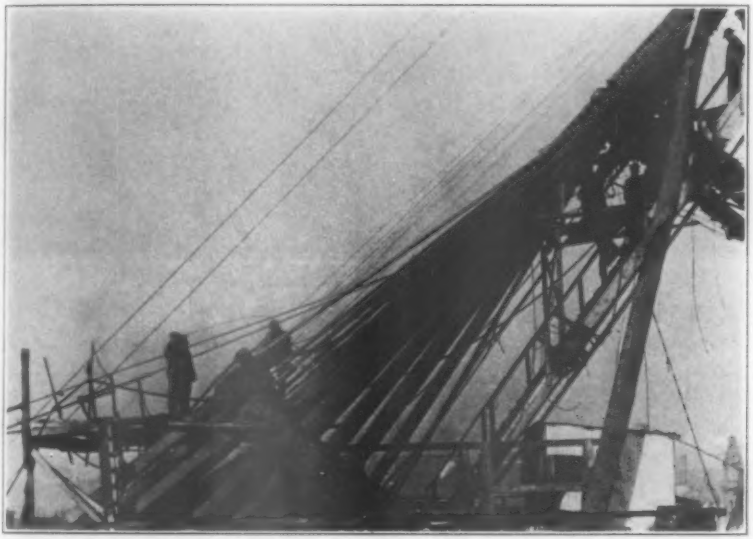
Philadelphia anchorage, north cable—Removing shoe from strand-leg.

of the strand. It consists of two flat bars which rest against the square end-face of the strand-shoe and straddle the two parts of the strand. These bars are bolted together with filler plates between them to serve as spacers and bearing for the strand-shoe and, by means of turned pins at their ends, are connected to the square steel bridle-bars which engage the wire-rope pennant at their lower end by means of a turned pin. When ready to release the strand from its temporary anchorage on the strand-leg, the independent hand-pumps operating the pulling-jacks were started. As soon as the strand pressure was off the strand-leg, the latter was removed from the anchor-chain pin and one of the eye-bars was also slipped from this pin. The strand-shoe was then turned from the horizontal position it occupied during the stringing to

its final vertical position and placed on the pin with a predetermined thickness of shim plates between the pin-bearing block and the strand-shoe bearing. The eye-bar was then replaced on the pin and the pulling-jack strain released, leaving the strand-pull on the pin, but the " bridle " was not removed from the strand-shoe attachment at one anchorage until the adjustment of the strand was completed across the span working from that anchorage as shown in Fig. 12.

At each main tower and on the inclined cable bents at the

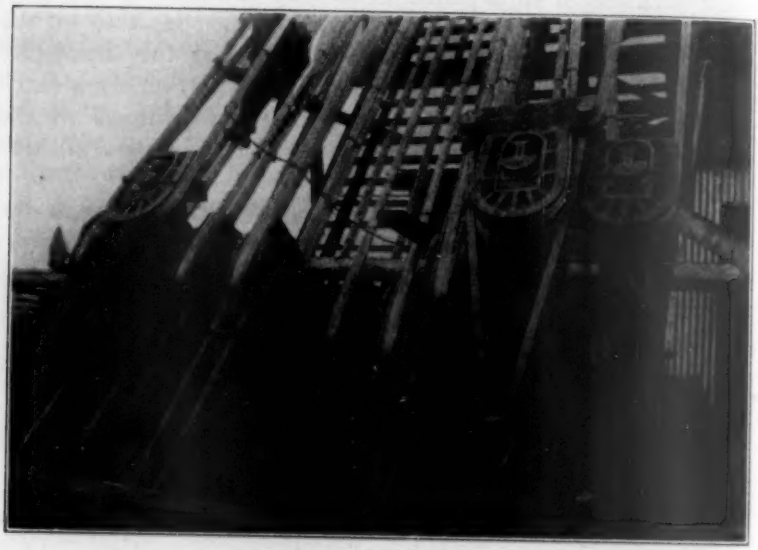
FIG. 12.



Philadelphia anchorage. View of north cable.

anchorage, the same strand was connected to what is termed a " balance-beam " as illustrated in Fig. 13; this consisted of two channel-beams of suitable length and strength for the load to be lifted. These beams were riveted together with a space of ¾ inch between the webs for the insertion of the lifting links which were connected to the " balance-beam " by pins through the webs. The " lifting links " were of varying length to conform to the curve of the cable in the saddles and were attached to the strands by means of a loop of ½-inch-diameter steel rope and clevises. It had been planned to use flat-wire seizings as had proven quite successful on the smaller strands of the Bear Mountain-Hudson

FIG. 13.



Cable strands at eye-bars.

FIG. 14.

North cable at Camden bent—placing strand in saddle.

River Bridge, but the load proved too great and the adjustment to equal stress too difficult for their successful application. Consequently, after several trials and failures to accomplish the work, they were abandoned in favor of the small rope lashings. Previous to these two bridges, round-wire lashings had always been used for laying the strands in their places in the cable saddles and had always proved more or less a source of trouble on account of the difficulty of getting the wires removed from under the strands when resting in the saddles. With the strands continuously served with soft-annealed, very thin, flat-wire seizing between the temporary sheave bearings, instead of the hemp twine previously used for keeping the wires of strands in their true position, there was very little trouble to keep the strands in their circular form and lay them in their true position in the hexagonal-cable saddle-groove as illustrated in Fig. 14.

REGULATION OF THE STRANDS.

As soon as the first strand had been attached to its pin at the anchor chain and had been lowered into its groove in the cable saddle at the inclined bent and the main tower, its regulation to true sag elevation in that land span was begun. The operation here was performed in precisely the same manner as had been employed for getting the guide-wire length as previously described, using the same graphs, engineers' transit and level, and standardized thermometer. After the land-span segment had been regulated to the desired sag by the movement of the strand in the main-tower saddle towards the land or river by means of steamboat ratchets attached to the strand near the ends of the saddle, as or if found necessary, the main-span segment was regulated in the same manner to its proper sag, and the strand was marked at the tower saddles so as to check possible slipping through the saddles. Then the opposite land-span segment was regulated, thus completing the operation of setting the first strand of the cable, provided there was enough leeway remaining at that strand-shoe for the insertion of the necessary amount of shims to make the correct length of strand as determined by the sag measurements. If there did not happen to be sufficient leeway in the slotted strand-shoe pin-hole at that anchorage-end to adjust the strand to correct sag in that side span, the strand would have to be adjusted back to the other side of the river where there was

leeway for shimming to get the correct length. This will happen at times in any cable construction, in spite of guide-wire measurements of the strand at the beginning of stringing. The tendency is always for strands when completed to be a little longer than the guide-wire length to which they are initially set.

After the completion of the first strand in place and the checking of sag elevations sufficiently to satisfy the engineers of its correctness, the instrument work was no longer required in setting the remaining strands. All the strands lying in the same horizontal layer in the hexagonal-shaped strand-cable were set to same sag and the next layer above was set, strand by strand, so that they touched the strands below without bearing hard against them. If any one strand was too long, it immediately showed in the work, by the spreading of the two adjacent strands below it. Frequently it became necessary to readjust strands after they had been satisfactorily set on account of misjudging the temperature of the strands before they were laid alongside of those already in the cable. The only time a reliable survey of position of the strands in the cable could be made was the early morning after they had arrived at a uniform temperature before the sun had begun to warm up the more exposed strands, or during a protracted period of cloudy weather.

After about twenty of the strands of each cable had been completed and regulated in their place, they would spread out so in mid-spans on account of the sun's rays heating the top strands and elongating them that they occupied too much area of the footwalks for the convenience of the workmen; therefore the superintendent in charge of the work devised a timber yoke having the hexagonal form of the strands in the cable with strand-grooves cut in the three faces. These yokes were applied at intervals of about two hundred feet by means of wire ties across the top over the strands already in place, and proved very effective in keeping the strands in their respective positions in the hexagonal section and in preventing further annoying spreading of the strands during the sunny part of the day. These yokes were extended after the strand construction had been completed to and including the middle row of nine strands, to form the inward sloping hexagonal sides above the middle of the cable.

In the construction and laying of the sixty-one strands in place in each of the cables, very little difficulty was experienced beyond

minor defects in equipment that were easily overcome and which could have been almost completely avoided had there been a little more time for preliminary study before the equipment was ordered.

THE SQUEEZING OF THE CABLE.

Immediately upon the finishing of the spinning and regulating of all of the strands in a cable, the squeezing of the sixty-one strands into the final cylindrical 30-inch-diameter cable was begun.

FIG. 15.

Cable-squeezing frame—north cable—main span.

Four hydraulic squeezers, as illustrated in Fig. 15, had been designed for this work. The squeezer consisted of a rigid frame of hexagonal form, composed of steel castings forming the six corners or angles to which the double-channel-iron hexagonal sides were bolted by suitable machine-turned bolts. These steel castings also formed the sockets into which were inserted the forged-steel cylinders of six hydraulic pushing-jacks, the plungers of which were 3½ inches in diameter. These plungers in turn were attached to cast-steel segments having a face radius of about one-half the diameter of the finished cable, *viz.*, 14¾ inches; these segments were provided with one end forked and the other single-ended so as to form a continuous ring of steel around the cable

when opened up to a maximum diameter of about 35½ inches before the application of the hydraulic pressure. Each squeezer was operated by three independent hydraulic pumps, each pump being connected by copper tubing, T's, and unions, to two of the six hydraulic jacks. These squeezers were designed to exert a pressure of 30 tons each, or a total pressure of 180 tons on the cable, at a working pressure of about 6250 pounds per sq. in. on the pumps. It would have been better had the plungers been

FIG. 16.

Erecting cable-band—Philadelphia side span—north cable.

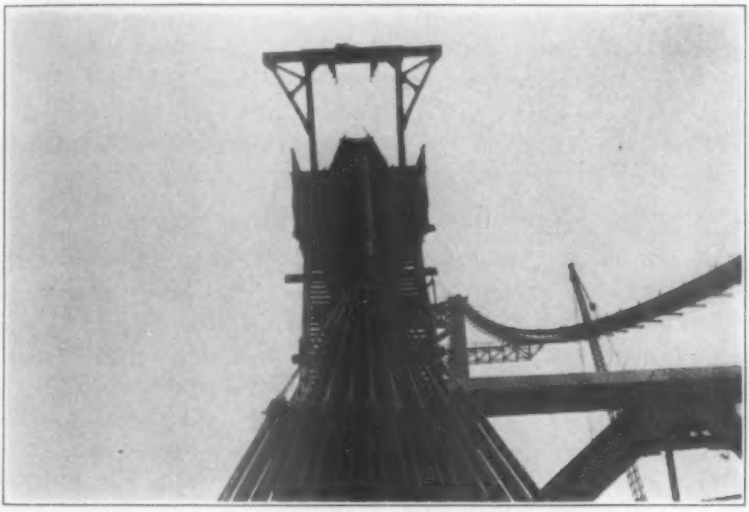
4 inches in diameter, requiring less elbow power on the pumps; but, all said and done, they were very satisfactory in operation both as to speed and work done. It had been planned to use round-wire seizings for the final binding of the cables, as had been the previous practice on smaller diameter cables; but the wire seizings composed of seven or eight turns of No. 10 galvanized wire would pop off as fast as applied, and therefore galvanized steel wire rope 9/32-inch diameter was substituted and applied at intervals of about two feet throughout the length of each cable.

PLACING THE CABLE BANDS.

Upon completion of the squeezing, the cable-band positions on the cables were located by careful steel-tape measurements along the cables, working from both towers and checking with the

computed length of the cable, also by transit check lines from one cable to the other at the middle of the river span. The cable bands were then applied in the usual way, working from main towers towards the middle of the main span and towards the anchorages in the land spans. The cable bands were hoisted to the tops of the towers by tower derricks, and were skidded along the walk on sleds made for the purpose, taking a half-band at a time, the heaviest of which weighed about 1700 pounds. The bands were

FIG. 17.



Splay collars—north cable at Philadelphia anchorage.

placed on the cables by means of a gallows-frame, straddling the cable, from which were suspended two differential chain-hoists as shown in Fig. 16. The two half-bands were hoisted separately, and then brought together and enough bolts inserted to hold the band in place. Another crew of men followed along, inserting the remainder of the bolts, and a third crew followed, screwing up the bolts with socket-ratchet wrenches. By an examination of the photograph it will be observed that at the point where the half-bands are being hoisted, there are two seizings that are nearer together than the others. These seizings were exactly located so as to come under two grooves provided on the inner face of the cable bands to receive them, and their purpose was to hold the wires in place against possible dragging while placing the bands.

It is the speaker's belief that their use here, and on the Bear Mountain Bridge for the first time, was eminently satisfactory and worth-while. (Fig. 17.)

LASHING THE FOOT-WALKS TO THE CABLES.

Upon the completion of the placing of the cable bands, the work of lashing the foot-walks to the main cables was begun and continued to completion, and then the storm cables and pull-back guy-ropes were removed. These were all cut loose at their end-fastenings, hoisted up and laid along the foot-walks, and later dragged over the walks to the Camden land span, where they were lowered to the dock on the north side of the bridge, cut up into suspended lengths and socketed. The three cross-bridges of the main span and the two in each land-span foot-bridge were then lowered to scows in the river and, in the case of the Philadelphia land span, upon the dock underneath the bridge, and dismantled.

REMOVING THE FOOT-BRIDGE ROPES.

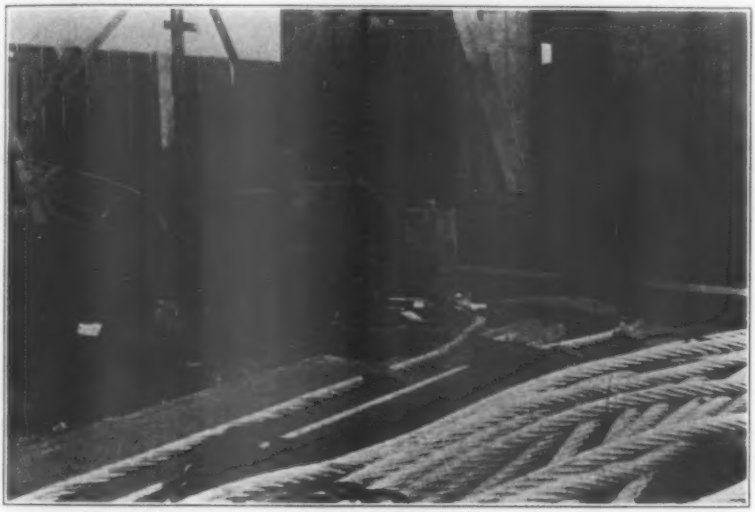
Now with the foot-walks suspended from the main cables, the main foot-bridge cable-ropes were disconnected at the anchorage-ends, and were hoisted upon the walks and lowered, one by one, to the Camden dock before-mentioned, where they were also measured and cut to lengths and socketed for suspenders. All the 2¼-inch-diameter suspender rope used for the temporary works of the main-cable construction furnished about 61,000 lin. ft. of finished suspenders for the main superstructure; the remainder of the approximately 105,000 lin. ft. required was cut from 44,000 lin. ft. of new rope, ordered and furnished under contract No. 13 of the Bridge Commission, which contract also included the main cable bands and fastenings and the two lines of 1-inch-diameter strand hand-rail ropes over each main cable with their connections.

MEASURING, CUTTING AND SOCKETING THE SUSPENDERS.

The work of measuring, cutting and socketing of the suspenders was done, as before mentioned, on the Camden dock where a straight and level runway, long enough to accommodate the longest loop of suspender, was constructed of timber planking. At the shore end of this runway, a form was made to conform to the profile of the suspender groove around the main cable bands and the drawing together of the two parts of the suspender loop a fixed distance below the cable (from which point they hang

parallel to the connection on the vertical post of the truss). In measuring the suspenders, they were laid around this form, as shown in Fig. 18, stretched out along the runway in the two parallel lines, and put under sufficient strain to pull them straight, and then measured to their respective computed unloaded lengths. The ends were then seized in the usual way, and the ropes cut to the measured lengths by hydraulically operated shears. The button-end sockets are shown in place on ropes in Fig. 19. The

FIG. 18.

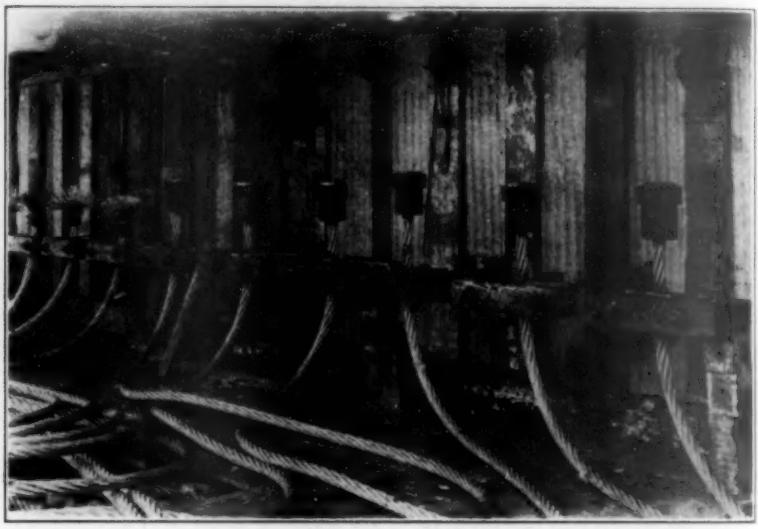


Templet for cutting suspenders.

socketing was done in the usual way, except that, in place of wire seizings around the rope adjacent to the socket to prevent the unlaying or untwisting of the strands of the rope, two special rope-clamps, as shown in the photograph, were used. The wires of the strands were untwisted, separated and straightened, then wiped and cleaned of all grease possible by use of kerosene oil. The end of the rope, then looking like a stiff bristle broom, as illustrated in Fig. 19, was dipped in a mixture of equal parts of muriatic acid and water until all traces of grease and most of the galvanizing had been removed. Then it was dipped and washed in clean water and wiped dry before being placed in the timber clamps shown in Fig. 19. The wires were then bound compactly together at the end by a temporary wire seizing, and the cast-steel

button-end socket with conical opening or basket was placed on the rope, with the ends of the wires flush with the end of the socket when the wire seizing had been cut away and the wires spread out and evenly distributed to fill the "basket" opening. Fire-clay or other suitable damming material was then applied around the rope at the small end of the socket, to prevent leakage of the spelter, and the socket was poured full of melted zinc to a finish projecting about ¼ inch beyond its end, as shown in Fig. 19.

FIG. 19.



Suspender ropes—socketing frame.

PLACING THE SUSPENDERS.

As the ropes were socketed, they were labelled with metal tags wired to them, indicating their position on the cables, and were then laid on the deck of a scow moored along the front end of the dock. The scow was then towed along under the spans, and the suspenders were hoisted to their respective places by a single line leading from the top of the main tower through a gate-block hanging from the cable band. The free portion of this line was clamped to the suspender rope a little beyond its middle point, at the end next to the socket, and at an intermediate point, with hemp mousings at intervals of a few feet between the main clamps. As the suspender rope was hoisted up and laid in the groove over

the cable band, the clamps and mousings were removed and the rope was allowed to fall down on the other side of the cable until its middle point was over the centre of the cable. The suspender clamps were then put on the loop of rope about four feet below the cable, completing the hanging of one pair of the suspenders. The short suspenders along the middle of the main span were handled two at a time, or all the four suspenders at a cable band were placed in one lift.

PLACING THE HAND-RAIL ROPES.

The four one-inch-diameter strand hand-rail ropes, previously cut and socketed to the computed lengths, were attached to their connections at the main-cable saddles on the inclined cable bents and on the main towers, adjusted to the desired sags in the three spans, and attached to the ¾-inch-diameter rod-stays connecting them to the main cable bands.

DISMANTLING THE FOOT-WALKS.

As soon as the suspenders had all been placed on the main cables, the cables and suspenders painted, and the cable bands, hand-rail ropes, and attachments completed, the main foot-walks and all other temporary works were removed. The foot-walks were taken apart and, as far as possible, lowered to the deck of scows in the river.

HANDLING THE WIRE AT THE BRIDGE SITE.

One main point of difference in the operations on the Delaware River Bridge cable-work from recent previous cable construction was that of handling the main-cable wire. Instead of the wire being spliced at the place of manufacture before shipping to the bridge site on drums or reels, it was shipped from the manufacturer's plant in the five-foot diameter coil lengths of approximately 3000 feet, and stored in the long dock-shed back of the Camden anchorage. In this shed were installed the necessary threading machines, swifts, reeling machines, oil-bath tanks, etc.; and the wire was there spliced, oiled, and placed on the 80,000-foot capacity cable-spinning reels, as shown in Figs. 20 and 21. For the strands of the main cable that were strung from the Camden side, these reels were delivered by small railway trucks to the anchorage, and hoisted to their places in the reel-stands by the derrick on the anchorage. The reels of wire for the

FIG. 20.

Cable wire threading machines at Camden reeling plant.

FIG. 21.

Wire reeling plant, Camden.

strands strung from the Philadelphia side were loaded on a motor truck, ferried across the river, and delivered in the reel-stands by the stiff-leg derrick on top of the Philadelphia anchorage.

The appliances for splicing and reeling of the wire, designed and installed by the American Cable Company in a building on the Camden side of the river, were the most complete and efficient ever used on cable building, and reflect great credit to the designer. They never failed once to meet the demand for wire on the work.

HANDLING OF THE FIELD ORGANIZATION.

As the time was limited for the completion of the bridge cables, how best to accomplish the task was a question left to the constructor to solve. The plant was designed, in the light of previous experience, to produce economically the maximum output of wire in place in the cable, and the method of accomplishment was left to the superintendent in charge of the work for the constructor. The superintendent chose to do the work with one twelve-hour shift per day, instead of two or more shifts of eight hours each. Flood-lights were provided for the early morning and late evening hours (6:00 A.M. to 6:30 P.M.) and signal lights, telephones and signal bells were provided for use in signalling the wire-adjusting orders both during the hours of darkness and during foggy weather. The smoothness and calm with which the work proceeded after dark, and the general morale among the workmen during the severely cold weather with night work, proved the wisdom of the Superintendent of Construction. Although the hours were very long, most of the work was not too arduous for such hours and the workmen all seemed satisfied. It was a rather severe test of physical endurance and stamina at times when the weather became severe, but there was not a single desertion to the speaker's knowledge, in spite of frequent frost-bitten fingers, etc.; the men were most loyal to the work.

APPLICATION OF FLAT-WIRE SEIZINGS.

Previous to the construction of the Bear Mountain-Hudson River Bridge and the Delaware River Bridge, all the principal parallel-wire-cable suspension bridges having more than seven strands per cable were built with what is termed a "core-cable." This was chiefly due to the fact that the strands of the cables had always been temporarily seized, or had their wires bound together

at intervals of a few feet, with a band of several turns of soft annealed steel or iron wire, the ends of which were twisted together. As it was necessary to cut apart and remove these temporary seizings, the work became difficult to accomplish with many strands in the cable. Consequently, after the cables had been spun beyond the seven central strands, the strands were wedged apart by wooden wedges driven between them, all the strand seizings were removed from the seven central strands and these seven strands were squeezed into a solid cylindrical core-cable around which permanent wire seizings of a small gauge wire were put on at intervals of two feet or so. This work was always a source of delay, and it was always doubtful whether all the wire seizings had been removed from the individual strands before squeezing. With the number of strands increased to sixty-one in the Delaware River Bridge cables, while thirty-seven strands were the largest number used in any previous cable construction, the problem became much more complicated. The study of this problem led to the invention and adoption of the flat-wire seizing which is illustrated in Fig. 22. These bands are made of flat or ribbon-wire, ¾ inch wide and about 1/100 inch thick, having a sufficient breaking strength to perform the task of firmly binding the wires of strand together during the cable stringing and soft enough to readily stretch and bend or break while the cable is being squeezed into the final cylindrical cable-section. Preliminary tests of a sample seven-strand cable with these seizings on the individual strands proved their efficacy and the use of the flat-wire seizings was written into the specifications for Delaware River Bridge cables.

In the meantime the Bear Mountain-Hudson River Bridge construction was begun and the use of the flat-wire seizings on the cable spinning was requested and adopted. Its successful application on that work with a 37-strand cable, where no squeezing was done until all the strands were completed in place, gave pretty good assurance that it would be successful on the 61-strand cables.

A TEST SECTION OF THE CABLE.

The cable contract provided for the construction of a full-sized section of the cable to test the diameter for boring the cable bands and for other tests. This sample cable was made with the sixty-one strands of 306 wires each, five feet long; each strand had

the theoretical number and spacing of wire splices and was seized with the flat-wire seizing. The strands were laid up in hexagonal yokes to conform to the position of the strands as laid in the cable when built.

One of the hydraulic cable-squeezers designed for the work was then applied to the sample cable; it was squeezed to the minimum diameter and seizings of small (9/32-inch diameter) wire rope applied. The cable band already bored to the computed diameter of 30 inches was then placed on the cable. From this test it was determined to bore all the cable bands for the cables to a finished diameter of 29-15/16 inches.

TIME REQUIRED FOR CABLE CONSTRUCTION ON VARIOUS BRIDGES.

A comparison of time in building the cables of various bridges beginning with the Brooklyn Bridge, and a brief outline of the principal factors entering into the cause of the great differences, may be of interest here.

COMPARISONS IN TIME OF CONSTRUCTION OF CABLES.

Comparative time-records for the cable construction on various large suspension bridges are as follows:

Brooklyn Bridge: Four cables, 15¾-inch diameter—5358 wires, .187-inch diameter in each cable. Total, 3500 tons. Construction of the foot-walks—ten months from laying the first rope across the span to spinning the first wire. Spinning the cables—twenty-one months. Maximum tons of wire spun in a day—nineteen and one-half tons.

Williamsburgh Bridge: Four cables, 18¾-inch diameter—7696 No. 6 wires in each cable. Total, 4500 tons. Construction of the foot-walks—seven months. Spinning the cables—seven months. Maximum tons of wire spun in a day—seventy-five tons.

Manhattan Bridge: Four cables, 20¾-inch diameter—9472 No. 6 wires in each cable. Total, 6300 tons. Construction of the foot-walks—four months. Spinning the cables—four months. Maximum tons of wire spun in a day—130 tons.

Delaware River Bridge: Two cables, 30-inch diameter—18,666 No. 6 wires in each cable. Total, 6500 tons. Construction of the foot-walks—three and one-third months. Spinning the cables—five months. Maximum tons of wire spun in a day—100 tons.

Bear Mountain-Hudson River Bridge: Four cables, 18¾-inch diameter—7252 wires in each cable. Total, 1900 tons. Construction of the foot-walks—one and two thirds months. Spinning the cables—two and one-half months. Maximum tons of wire spun in a day—76 tons.

Some of the factors contributing to this recorded improvement in time-records for cable construction are indicated in the following descriptive account of the evolution of plant and methods as applied to the successive large suspension bridges.

EVOLUTION OF PLANT AND METHODS FOR CABLE CONSTRUCTION.

The foot-bridge used in construction of the Brooklyn Bridge cables consisted of a single pathway four feet wide, supported on two foot-bridge cables 2½ inches in diameter, extending from anchorage to anchorage over the towers and lying parallel to the strands during the spinning. This foot-walk was for the use of the wire regulators in gaining access to what was termed the "cradles." The "cradles," one at the middle and one at each quarter-point of the main span, and one at the middle of each side span, were transverse walks, four feet wide and about eighty feet long, supported by three 2¼-inch-diameter ropes. These "cradles" were used by the wire regulators during the spinning of the strands; they also carried the supports for the hauling-rope sheaves. There was no other point of access to the strands during the spinning except at the anchorages and at the main towers. After the spinning of a strand was completed, the wire seizings were applied by men standing in a buggy hanging from sheaves running on the strand itself. The strands were then regulated to their unloaded sags in the main cables, and their middle points were at a considerably lower level and entirely inaccessible except by a buggy running on the strands. The spinning plant was located on the Brooklyn anchorage and consisted of two loops of hauling rope, each with its 50-h.p. engine, traction wheel and idlers, and each serving two cables. A part of the hauling rope and wire-spinning wheel or sheave lay above each main cable, and the wire was all spun from the Brooklyn side, the sheave returning empty to that side while the wheel on the other part of the loop was hauling a bight of wire to the New York side for the strand it was spinning.

Briefly described, the foot-bridge used in the construction of the Williamsburgh Bridge cables consisted of a double-deck structure in the main span and a single-deck in each land span, affording access to the strands of each of the four cables throughout their length during both the spinning and the squeezing. The spinning machinery and the loops of hauling ropes were arranged as for the Brooklyn Bridge, but the travelling sheaves were designed for spinning the wire in both directions, and the wire was spun from both sides of the river, thus doubling the capacity.

On the Manhattan, Bear Mountain, and Delaware River Bridges, the foot-bridges consisted of a single-deck walk under each cable, giving access to the strands throughout their length during the spinning and to the main cables for all other work.

The spinning machinery and loops of hauling ropes were an independent plant for each cable, thus doubling the capacity of performance per cable as compared with the Williamsburgh Bridge plant, and quadrupling the capacity per cable as compared with the plant employed on the Brooklyn Bridge cables.

In any comparison of the respective time-records, it should be noted that the Delaware River and Bear Mountain Bridges have but two cables each, thus halving the hourly capacity for cable spinning as compared with the earlier large suspension bridges which were built with four cables.

ACKNOWLEDGMENTS.

I wish to express here my appreciation of the loyalty, coöperation and aid rendered by the Assistant Engineers of the Board of Engineers, who were assigned to the cable construction work; and of the confidence placed in me for my part in the accomplishment by the Chief Engineer and members of the Board of Engineers.

Acknowledgment should also be made here of the tireless services, skill and experience, of Mr. L. N. Gross, the Superintendent of Construction for the Keystone State Construction Company, and his organization of loyal assistants, foremen and other workers.

I desire also to extend my thanks to The Franklin Institute, its Secretary, and to the Philadelphia Section of American Society of Civil Engineers for the honor conferred in asking me to speak at this joint meeting.

The Explosion in the Liquid Air Plant at Boulogne-sur-Seine. GEORGES CLAUDE. (*Comptes Rendus*, Feb. 8, 1926.)—For two years the author of this article had been collaborating with André Ribaud in the production of xenon and krypton from liquid air residues. These gases are acquiring importance in the technique of radiology. Since they occur to so small an extent in air, it is from the final evaporation of liquid oxygen that they are to be obtained. The process employed had been followed fifty times in more than one laboratory. Standing beside a container in which oxygen was evaporating, Mr. Ribaud was so badly injured by an explosion of the apparatus that he died in a few hours. The Government of France conferred upon him posthumously the cross of the Legion of Honor.

Such accidents, says the writer, are by no means unknown in the oxygen industry. "Without apparent cause after months or years of perfect operation an apparatus explodes, sometimes very violently." He attributes them in general to the accumulation of impurities from the large quantity of air liquefied. "No matter to how small an extent a gaseous impurity is present in the air treated, if it can be condensed by intense cold and if, further, it is much less volatile than oxygen, it collects in the bottom of the apparatus through evaporation. If, in addition, this impurity is combustible, all the conditions for a violent explosion are present by a singular fatality. Now such is the case with acetylene in particular." With the plant treating 1000 cu. m. of air per hour throughout a month, one part in a million of acetylene would result in the storing of 4 kg. of a very violent explosive. It may be that ozone acts as the detonator, for the author has found it to react with solid acetylene, or perhaps electric sparks due to the evaporation of the liquid oxygen are to blame. In any case the air condensed should be as pure as possible and the part of the apparatus containing the accumulated impurities should be buried in the earth. G. F. S.

Death-dealing Sound Waves. (*The Science News-Letter*, 8, No. 265, May 8, 1926.)—A "death-noise," instead of a "death-ray," was the phenomenon discussed recently before the National Academy of Sciences by Prof. R. A. Wood and Alfred L. Loomis, of the Johns Hopkins University. The "death-noise" would have been inaudible to human ears, but it consisted of sound waves just the same, and it killed small fishes and other aquatic animals in vessels of water, in less than a minute.

The two researchers generated exceedingly high-frequency sound waves by means of electrical apparatus. The waves were produced at a rate of from 100,000 to 400,000 to the second; the upper limit of audibility to human ears is between 20,000 and 30,000. If a beam of these sound waves is directed toward the surface of the water, Professor Wood stated, the surface is heaped up in a mound. The vibrations heat the water, a rise of nearly ten degrees Fahrenheit in one minute having been recorded.

HYDROGEN SPECTRUM LINES IN THE STARS AND IN THE LABORATORY.*

BY

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BARTOL RESEARCH FOUNDATION
Communication No. 3.

THE Balmer series of hydrogen, the simplest spectral series of the simplest of terrestrial elements, has been the subject of many experimental investigations. There are several reasons for this interest in the series entirely apart from its fundamental importance in the theory of atomic structure. The Balmer lines are the most prominent features of the spectra of many stars; in laboratory experiments the individual lines of the series undergo striking changes with experimental conditions; and the colors displayed by the tubes of luminous hydrogen are among the most beautiful in spectroscopy. The present paper describes the spectra of the hydrogen stars and brings to attention certain laboratory experiments with hydrogen spectra which have particular bearing on the interpretation of the stellar spectra.

The Balmer series begins with H_α in the red and is made up of lines which become rapidly closer together and weaker in intensity, with a theoretical limit in the near ultra-violet. The first four lines, H_α, H_β, H_γ, and H_δ, occur in the visible spectrum at wave-lengths, 6563.04, 4861.49, 4340.66 and 4101.90 Å., respectively. H_{20} is at wave-length 3676.51 and H_{30} at 3662.36 Å.; the limit calculated from Balmer's [1] formula is 3646.13 Å.

SPECTRA OF HYDROGEN STARS.

In Figs. 1, 2 and 3 are given reproductions of a variety of stellar spectra containing hydrogen lines. The spectra of Fig. 1, photographed by Sir William Huggins,[2] are very fine examples of the Balmer absorption series; indeed, the series was termed the "Huggins series," after its discoverer, although the name has perhaps been lost in the years. All the hydrogen lines of these

* Delivered Friday afternoon, March 12, 1926, in the lecture hall of the Institute.

[1] BALMER, *Ann. d. Phys.*, **25**, 80 (1885).

[2] HUGGINS, "An Atlas of Representative Stellar Spectra," Plate IX (1899).

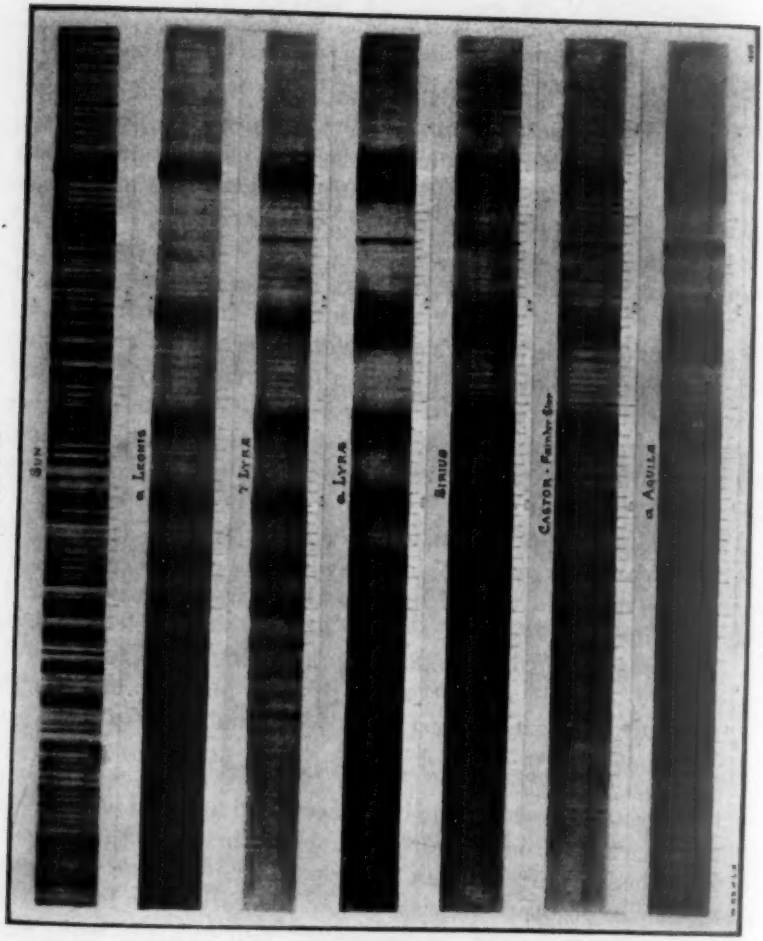
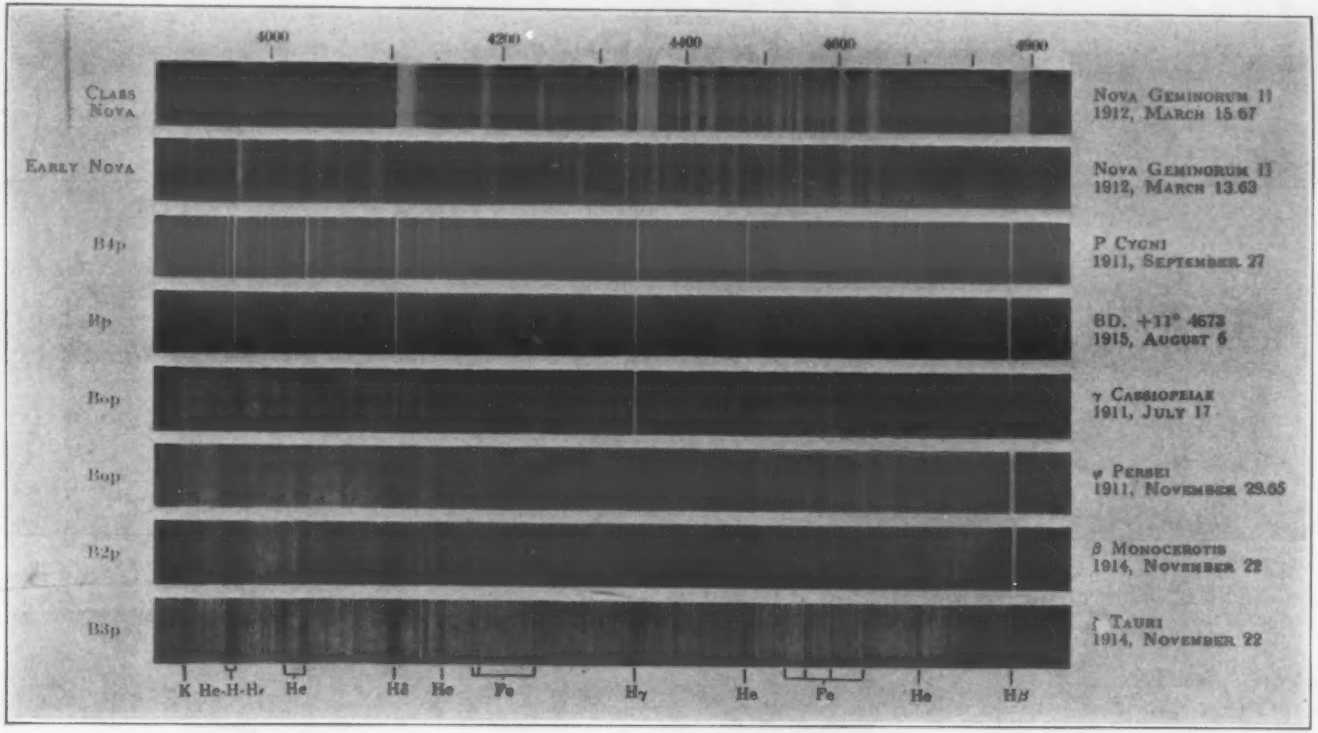


FIG. 1.

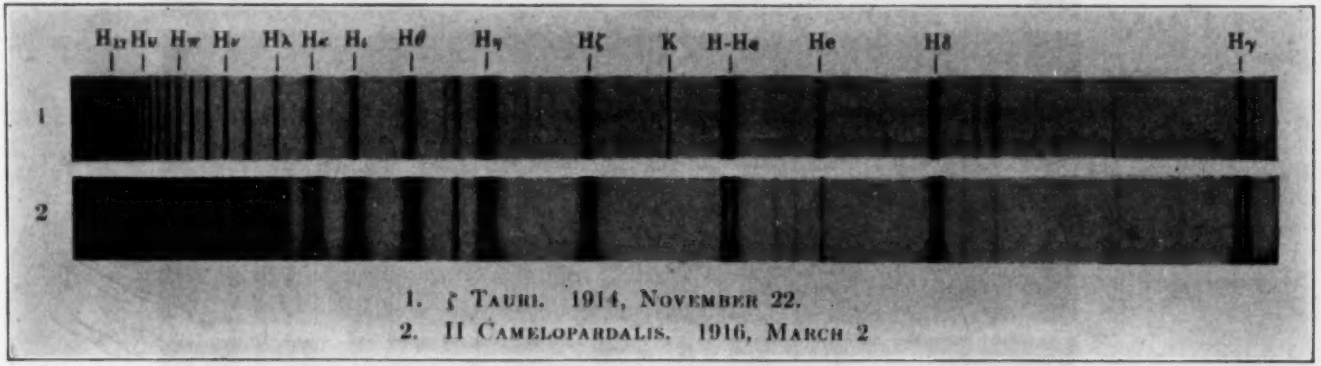
Stellar spectra with Balmer absorption lines.

FIG. 2.



Stellar spectra with various types of Balmer lines.

FIG. 3.



Stellar spectra: (1) Balmer absorption series; (2) Balmer emission series on absorption.

spectra are simple absorption lines, some are very broad. The stellar spectra of Figs. 2 and 3, due to Curtiss,[3] exhibit many types of Balmer lines both in absorption and emission. Some of the lines are exceedingly complex, as for example the triple reversals of H_β and H_γ in φ *Persei* and β *Monocerotis* (Fig. 2). These are each a broad, dark absorption band on a bright background with a narrower bright centre band which in turn is divided by a yet narrower line of absorption. These and other details of the lines of the stellar spectra will be referred to in subsequent paragraphs. As many as thirty-two members of the Balmer absorption series may be counted in certain stellar spectra. Twenty-seven absorption lines are visible in the spectrum of ζ *Tauri* (Fig. 3). The Balmer emission lines do not appear to be brought out in the spectra of the stars to the same extent as the absorption lines, it being rarely possible to distinguish as many as twenty-five bright members. The spectrum of *11 Camelopardalis* (Fig. 3) shows possibly twenty-five emission lines, although in this spectrum the bright lines are not simple emission lines but the bright cores of absorption lines. In the "flash spectrum" of the solar corona as many as thirty-five members of the emission series have been observed.[4]

THE BROAD BALMER LINES IN THE LABORATORY.

In the stellar spectra the lines of the Balmer emission series are for the most part relatively narrow, whereas the lines of the absorption series are often of extraordinary width, appearing as broad, dark bands across the brighter background of the spectrum. Why this is so has for long been obscure, but the explanation now begins to emerge as the result of the investigations of the laboratory. Curiously enough no one has ever succeeded in producing in the laboratory broad Balmer lines in absorption, whereas broad emission lines may be produced quite readily. Such wide emission lines are rarely observed in the stars.

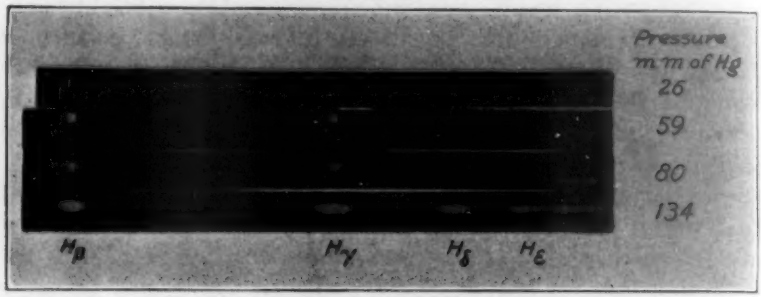
To obtain broad emission lines discharge tubes are required filled with hydrogen at a considerable pressure, of several hundred mm. of mercury, excited by heavy electric currents, such as the discharges of a condenser. This has been known for a long time. In the case of low pressures of hydrogen under more moderate

[3] CURTISS, *Pub. Univ. Mich.*, Vol. 3 (1921).

[4] EVERSHED, *Phil. Trans. Roy. Soc.*, 201 A, 457 (1903).

excitation by a transformer or induction coil, the Balmer lines are of course very narrow and weak in intensity. Illustrations of the broadened lines [5] are given in Fig. 4, which shows spectrograms of hydrogen excited by condenser discharges with the iron

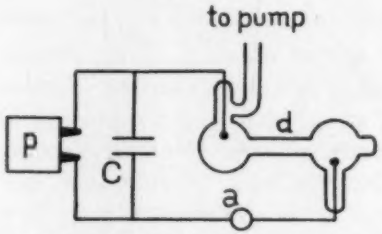
FIG. 4.



Balmer hydrogen lines broadened by pressure.

arc comparison spectrum. The pressures were 26, 59, 80, and 134 mm. of mercury and the currents 1.6, 2.0, 2.6 and 2.8 amperes, respectively. The arrangement of apparatus is given in Fig. 5, where p was a 30-kv., 1-kw. transformer, C a condenser

FIG. 5.



Hydrogen discharge tube and connections.

of capacity 0.01 μF, a a thermogalvanometer, and d a small discharge tube of pyrex glass.

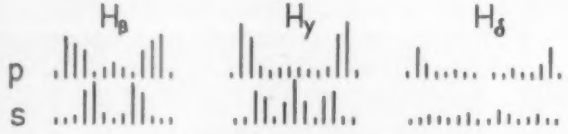
As the result of a number of investigations [6, 7, 8] the widening of the Balmer lines has been shown to be due to the presence of electric charges in the vicinity of the luminous atoms. In other words, the widening is a manifestation of the Stark effect.

[5] HULBURT, *Astrophys. J.*, **55**, 399 (1922).
[6] MERTON, *Proc. Roy. Soc.*, **92**, 322 (1915).
[7] NICHOLSON and MERTON, *Phil. Trans. Roy. Soc.*, **216**, 458 (1916).
[8] HULBURT, *Phys. Rev.*, **21**, 24 (1923).

We must mention that a spectrum line may owe its width to a number of causes,[9] such as atomic collisions, the Doppler effect of temperature agitation of the luminous atoms, etc. These, however, produce only a small widening, usually less than 1 Å.[10, 11] In the stellar spectra the widths of the hydrogen lines are often of quite a different order of magnitude, 20 or 30 Å., and therefore some much more energetic cause of the widening is evidently at work. As has been said, this is to be found in the Stark effect.

Stark [12] subjected luminous hydrogen to a powerful electric field and found that each Balmer line was split up into a large number of component lines symmetrically displaced to either side of the parent line. Furthermore, the intensities of the Stark components were equal on either side. This displacement in wavelength (or frequency) from the parent line was found to be closely

FIG. 6.



Stark effect components.

proportional to the strength of the electric field. The *p* and *s* components of the electrical resolution of H_β, H_γ and H_δ as determined by Stark are given in Fig. 6. The *p* and *s* components are polarized with electric vector parallel and perpendicular, respectively, to the lines of force of the external electric field. In the present discussion matters of polarization do not enter.

The picture of the hydrogen gas which is emitting broad Balmer lines is as follows: Intermingled with the radiating hydrogen atoms in the discharge tube are charged particles, such as electrons, or ions, or perhaps even the component charges of a neutral atom. As a result the radiating atom finds itself in an electric field and, therefore, instead of a single line it emits a group of lines, the Stark components, whose displacements are proportional to the strength of the electric field. Some atoms being very close

[9] RAYLEIGH, *Phil. Mag.*, **29**, 274 (1915).
[10] FABRY and BUISSON, *Jour. de Phys.*, **2**, 442 (1912).
[11] MICHELSON, *Astrophys. J.*, **2**, 251 (1895).
[12] STARK, *Ann. der Phys.*, **48**, 193 (1915).

to charged particles will be in an intense electric field; their Stark components will be far from the parent line. The components of other atoms, which are relatively far from charged particles, will be but slightly removed from the parent line. The radiations of all the atoms together make up the broadened line which is observed in the spectroscope.

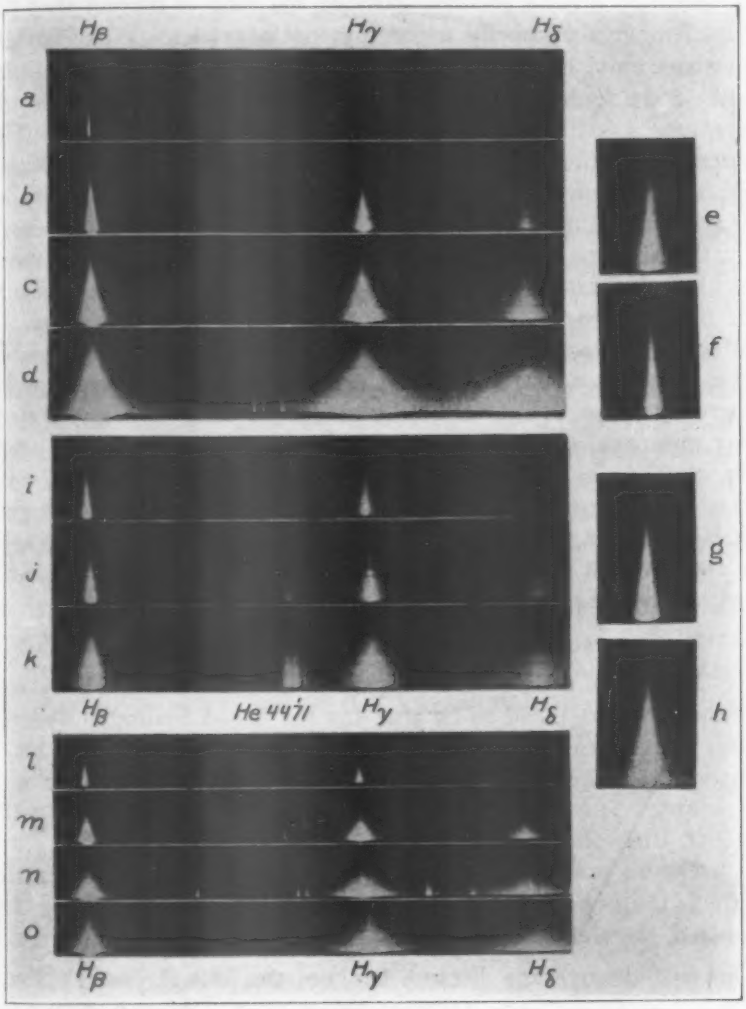
In order to verify these ideas quantitatively a mathematical expression was derived [8] for the distribution of intensity across the widened line. The derivation involved the assumption of a probability law for the distribution of atoms and ions or electrons and an inverse square law for the strength of the field, and the introduction of Sommerfeld's [13] quantum expression for the Stark displacement. The intensity distributions across the broadened lines were then measured [8] for H_β, H_γ and H_δ, by photographing the spectra through a neutral wedge. Such spectra are given in Fig. 7, *a*, *b*, *c*, and *d*, which were for hydrogen at pressures 48, 112, 160 and 250 mm. of mercury, respectively. Condensed discharges were used, the arrangement of apparatus being that of Fig. 5 and the respective currents were 1.2, 2.3, 2.8 and 4.1 amperes. From the contour of the line the intensity distribution across the line was determined and was found to agree closely with the theoretical expression. The broadening may therefore properly be termed the "Stark broadening" or the "Stark quantum broadening." It is seen, then, that the increase of pressure or current causes a wider line merely because the numbers of ions and luminous atoms increases with these factors and hence the probability of the luminous atom being in an intense electric field also increases.

Further experiments were performed to test the correctness of the Stark broadening theory. For instance, an inductance in series with the hydrogen tube should reduce the width of the lines. For the inductance distributes the condenser discharge over a longer interval of time, so that the pulses of current are of less intensity. This was found to be true, as shown by *g* and *h* of Fig. 7. These are photographs of H_β with the inductance in and out, respectively, the current and pressure being the same in each case. The neutral wedge was used because the relative widths of the wedge-shaped lines were easy to estimate. Again, a

[13] SOMMERFELD, "Atomic Structure and Spectral Lines," p. 286, 3rd Edition (1922).

quenched gap placed in series with the tube increased the width of the lines, as it should. Photographs e and f (Fig. 7) of H_β were

FIG. 7.



Neutral wedge spectra of Balmer lines broadened under various conditions.

taken with the gap in and out, respectively, the current and pressure being constant. The insertion of the gap enabled the charges to accumulate on the condenser plates to higher potentials, so that

when the discharge occurred a greater current pulse than otherwise passed through the hydrogen tube.

A further interesting verification of the Stark broadening was obtained by adding an inert gas. If the view is correct that the broadening is primarily a question of large density of charged particles amid the luminous hydrogen atoms, the narrow Balmer lines from hydrogen at a low pressure should widen when the pressure was increased by the addition of a foreign gas. The electrons and ions produced in this gas by the electrical discharge should be as effective as those of hydrogen. This was found to be true. The Balmer lines widened when the pressure was increased by the addition of helium, nitrogen,[8] oxygen, air, carbon monoxide and carbon dioxide,[14] in the same manner as by the addition of more hydrogen. Spectrograms *i*, *j*, and *k* (Fig. 7) show the effect of helium. The pressure of the hydrogen was 29 mm. in the three cases and of helium was 79, 188, and 380 mm. of mercury, respectively. Spectrograms *m*, *n* and *o* (Fig. 7) were for mixtures with nitrogen. The pressures of hydrogen were 85 mm. for *m* and *n* and of nitrogen were 0.55 and 115 mm. For *o* the pressure of hydrogen was 200 mm. with no nitrogen. It is interesting to notice the striking similarity of photographs *n* and *o* which were taken with the same total pressures but with quite different partial pressures of the component gases.

THE BROAD BALMER LINES OF THE STELLAR SPECTRA.

The Stark theory of the broad hydrogen emission lines was immediately assumed to be true for the broad hydrogen absorption lines in the spectra of the stars. By combining the Stark theory with the theory of high-temperature ionization developed by Saha,[15] it has been possible to estimate the conditions in the stellar atmosphere.[16, 17]

Assume pure atomic hydrogen gas ionized by its temperature. If ν is the number of atoms per c.c., of which a fraction α are ionized, there are $2\alpha\nu$ charged particles per c.c., each of charge e. One-half the average distance between the ions is $\frac{1}{2}(2\alpha\nu)^{-\frac{1}{3}}$ cm. Therefore from Coulomb's law of force each radiating atom finds

[14] HULBURT, *Phys. Rev.*, **23**, 106 (1924).
[15] SAHA, *Phil. Mag.*, **40**, 472 (1920).
[16] HULBURT, *Astrophys. J.*, **59**, 177 (1924).
[17] RUSSELL and STEWART, *Astrophys. J.*, **59**, 197 (1924).

itself in an electric field of mean value $4e(2\alpha\nu)^{\frac{2}{3}}$ electrostatic units. This calculation is of course very rough, but has been presented because of its simplicity. A more elaborate averaging [17] gives $6.95e(2\alpha\nu)^{\frac{2}{3}}$ for the mean field. According to Stark [18] in the case H_β the extreme components are displaced 20 Å. each way by a field of 1.04×10^5 volts, which amounts to a widening of 0.115 Å. per unit c.g.s.e.s. field. If w is the width of the line in Ångströms.

$$w = 0.8e(2\alpha\nu)^{\frac{2}{3}}. \qquad (1)$$

The discussion has borne upon the emission of radiation, but may be taken to refer equally well to the absorption of radiation.

Taking ν to be 2.7×10^{19} at 0° C. and atmospheric pressure, and replacing e by the value 4.77×10^{-10}, equation (1) becomes

$$w = 2 \times 10^5 (\alpha P T^{-1})^{\frac{2}{3}}, \qquad (2)$$

where P is the pressure in atmospheres and T is the temperature on the Kelvin scale.

Assuming the gas to be ionized by its temperature, the fraction α of atoms of hydrogen which are ionized is calculated from the reaction isobar equation for ionized hydrogen given by Saha.[15] For low pressures of the order 10^{-3} or 10^{-4} atmospheres and for temperatures above 5000°, α comes out close to unity, which is the value used in the following calculations. The pressure P may be taken as about 10^{-4} atmospheres, in accord with recent evidence, reviewed in a paper by Fowler and Milne,[19] which points unmistakably to low pressures in the reversing layers of the stars, of the order 10^{-3} or 10^{-4} atmospheres, or less.

Applying equation (2) to the case of the sun, taking $T = 5000°$ and $P = 10^{-5}$, gives $w = 0.3$ Å. The observed w for H_β is about 0.6 Å. Thus for the sun there is satisfactory agreement between the pressure calculated from the observed hydrogen line widths and that estimated in other ways.[17] In the cases of the stars with wide hydrogen lines the agreement is hardly so satisfying. For α *Lyræ* with $T = 10{,}000°$ and $P = 10^{-4}$, w is 0.9 Å. The observed width is easily 20 Å., and even 30 or 35 Å. if one considers the wide shallow wings of the absorption lines. For $w = 30$, P comes out 0.02 atmosphere. In short, the theory

[18] STARK, *Ann. der Phys.*, **48**, 202 (1915).

[19] FOWLER and MILNE, *Roy. Ast. Soc.*, Monthly Notices, **83**, 415 (1923).

indicates, and it is difficult to see how equation (2) can be in error by several orders of magnitude, that lines much narrower than 20 or 30 Å. will be produced by hydrogen at a pressure 10^{-4} atmospheres.

Fortunately, there are two factors (and perhaps others) entirely in accord with notions concerning the stars, which serve to reconcile wide lines and low pressures, first, reversing layers of considerable thickness, and second, large numbers of free electrons. The reversing layers are certainly extensive and in all probability are permeated with many electrons and ions which may come from the easily ionized metal atoms of the stellar atmosphere and from the thermal emission of the interior. Furthermore, the presence of a large number of electrons is a necessary consequence of the fact that the densities of all the stars are such that the average weight of the ultimate particles is low, of molecular weight between 2.0 and 2.5.[20] If the layer were of great depth the absorption would be sufficient to render effective the wide wings of the line which from a shallow layer would be inappreciable. If, referring to the calculations for α *Lyræ*, there were 200 times as many electrons as hydrogen atoms, the calculated line widths for $P = 10^{-4}$ would agree with those observed. Further, one would anticipate somewhat different types of absorption lines depending on which factor were operative. A wide absorption line caused by a relatively thick and cool reversing layer of low-pressure hydrogen would be expected to have a very intense and sharp central core with wide-spreading edges of weak intensity (provided the reversing stratum itself emitted no light, all the emission coming from the hotter interior in the background). Such lines are seen in the spectrum of α *Aquilæ* (Fig. 1). On the other hand, a shallower line, as those of α *Lyræ* (Fig. 1), would result from a thinner reversing layer either at a higher pressure or at a low hydrogen pressure with many foreign electrons or ions. Actually in general both factors coexist, and their experimental differentiation will be uncertain, except in extreme cases.

The fact that in the stellar spectra the Balmer emission lines are usually narrow, or if wide have central reversals, whereas the absorption lines may be very broad, now becomes intelligible. Wide absorption lines would apparently require relatively stable

[20] DREYER, *Roy. Ast. Soc.*, Monthly Notices, **84**, 554 (1923–1924).

stellar conditions for their production, namely, a bright continuous background spectrum from the hot interior of the star together with an extensive envelope of fairly cool hydrogen. Broad emission lines, on the other hand, would only arise from a layer of hydrogen atoms extremely hot (violently excited), either at a considerable pressure or permeated with large numbers of electrons and ions, with no reversing layer above it. Such a condition, if it existed, would manifestly be expected to be unstable, for the atoms from the hot layer must naturally diffuse outward to form a cooler absorbing atmosphere above. Apparently something like this happened to *Nova Geminorum II*. The hydrogen lines of the spectrum of this star (Fig. 2), which on March 13, 1912, were rather broad dark bands with a slight emission asymmetrically placed, suddenly changed on March 15th to broad emission bands with faint traces of absorption visible near the centres. The star may be imagined to have exploded into flaming hydrogen. After a few months the broad emission lines became narrower, some of them turning to absorption, until in 1914 the hydrogen lines were no longer very prominent features of the spectrum.

THE BALMER EMISSION SERIES IN THE LABORATORY.

Until a few years ago the Balmer emission series had never been produced in the laboratory beyond the twelfth member and the absorption series beyond H_β. At first sight this would seem strange because none of the simpler difficulties common to spectroscopic experiments are present. Pure hydrogen is readily obtainable and the Balmer lines occur in a spectral region positively ideal for photography, being entirely in the visible and near ultraviolet regions of the spectrum. Lenses, tubes and the like, of glass, not quartz, may be used in the apparatus. The trouble in bringing out the Balmer emission series is to be attributed to the very rapid falling off in intensity of the higher members and to the almost unavoidable appearance of the secondary and continuous spectra of hydrogen which mask the faint series lines. It is known that the Balmer lines are emitted by the atoms of hydrogen and the secondary spectrum by the molecules. Therefore the appearance of so many Balmer lines in the stellar spectra, and of so few in the laboratory hydrogen spectra, may be taken to mean that the concentration of atoms is much greater and of the molecules much less in the stars than in the terrestrial sources.

Furthermore, the physical conditions of the hydrogen of the stellar atmosphere differ from that of the usual vacuum tube in being at a lower pressure, and in being under intense bombardment by electrons from the hot interior of the star. These conditions would increase the number of atoms and decrease the possibility of the formation of molecules. Acting on this hypothesis, Whiddington [21] arranged a simple experiment to fulfill insofar as possible the conditions of low pressure and intense electron bombardment, and was able to record nineteen or twenty of the Balmer emission lines. A glass discharge tube was fitted with a hot cathode of tungsten wire. With the tube filled with hydrogen at a pressure of 10^{-3} mm. of mercury or less, and with the cathode glowing brightly, a current of several tenths of an ampere through the tube caused the radiations of atomic hydrogen to appear in great purity. Further, the relative intensities of H_δ to H_γ, of H_γ to H_β, and of H_β to H_α were enormously increased. Whiddington has published no photographs of the spectra which he obtained.

The experiment should be carried further; it offers enticing possibilities of repetition and development. The experiment is the first to indicate a way of varying at will the relative intensities of the Balmer lines by means of the pressure of the hydrogen and the energy of the bombarding electrons. A study of the intensities and the controlling factors would perhaps go far towards offering a clue to the intensity law of the series lines, a clue which considerations on Correspondence Principles and *a priori* probabilities have as yet failed to furnish. It might even be possible, by the use of low pressures and intense electron bombardments, say, to produce a Balmer spectrum with the long wave members of lower intensities than the shorter wave lines. Such experiments might be looked to aid in the interpretation of the changes in the intensities of the Balmer emission lines observed in the spectra of stars of Class *Md*,[22] of which *Mira Ceti* is the type star. These stars are long period variables. The bright hydrogen lines vary in relative intensity with the waxing and waning of the light; for example, H_δ sometimes appears twenty times as bright as H_γ, and later H_γ often reaches equality with H_δ.

[21] WHIDDINGTON, *Phil. Mag.*, **46**, 605 (1923).
[22] MERRILL, *Astrophys. J.*, **53**, 185 (1921).

The most complete extension of the Balmer series in the laboratory has been that of Wood [23] who photographed the series to H_{20}. In order to understand his method, we must call to mind what one would naturally do to obtain the series lines. One would use a discharge tube filled with hydrogen at a very low pressure and excited by powerful currents, on the idea that in some such manner a maximum number of radiating atoms and a minimum number of molecules would be produced. Such a straightforward procedure, however, has long been known to meet with no success. The secondary lines always appear in devastating intensity due to apparently unavoidable factors, namely, the walls of the tube and the metal of the electrodes. These somehow act as catalyzing agents to form molecules of the atoms produced by the electrical discharge. Wood avoided the effect of the electrodes by employing long discharge tubes with the electrodes placed in side branches at least 50 cm. from the central portion of the tube. The light was taken end-on from the central portion only. The catalytic action of the walls of the tube was almost entirely suppressed by mixing a little oxygen or water vapor with the hydrogen. Just why these gases prevent the glass walls from converting the hydrogen atoms to molecules is a mystery, although the fact has been known for a long time.

The Balmer spectrum obtained by Wood from the long tube is shown in Fig. 8; the print is a negative, the bright lines being dark. Currents of as much as 0.2 ampere, 60 cycles, were used and the pressure, about 0.5 mm. of mercury, was adjusted to the best condition. It is seen in Fig. 8 that the series lines became rapidly weaker, H_{20} being the last line to appear, until finally submerged by the lines of molecular hydrogen. Wood estimates the intensity of H_{20} to be 10^{-5} or 10^{-6} of H_{α}.

A further extension of the Balmer series by the method of long tubes and moist hydrogen appears difficult. For, even if the secondary spectrum were completely abolished by skill and care, there is still the veiling effect of the continuous spectrum to be dealt with. Just how this is to be done cannot be said, for the origin of the continuous spectrum is uncertain. It always appears in the light from hydrogen tubes, and is apparently unconcerned with the presence or absence of the atomic and molecular spectra.

[23] WOOD, *Proc. Roy. Soc.*, 97, 455 (1920); *Phil. Mag.*, 44, 538 (1922).

FIG. 8.

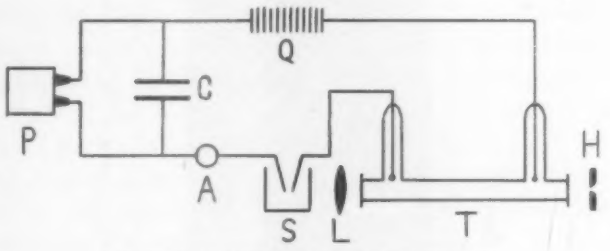
Laboratory Balmer emission series to $H\sigma$. The print is a negative.

Whether the continuous spectrum enters into Whiddington's experiment is not stated in his paper. The results of an experiment by Smyth and Brasefield [24] on the positive ray analysis of the ions in a discharge in hydrogen, taken together with recent spectroscopic information obtained in this laboratory, suggest that the continuous spectrum arises from the H_3 molecules.

THE BALMER ABSORPTION SERIES IN THE LABORATORY.

The Balmer absorption series has been observed in the laboratory only to the tenth Balmer line,[25] and that but recently. The method depended on sending a continuous spectrum through a tube of luminous hydrogen, the source of the continuous spectrum and the tube being lighted up in exact synchronism. The appara-

FIG. 9.



Apparatus for observing Balmer absorption lines.

tus is indicated in Fig. 9. The underwater spark S, with phosphor-bronze terminals, which gave a bright continuous spectrum in the region of the Balmer lines, was connected in series with the hydrogen tube T, so that both were lighted at the same time by the electrical discharges from the transformer p and the condenser C. The quenched gap Q was necessary to insure intense discharges. The glass discharge tube T, about 75 cm. in length between the electrodes, was filled with moist hydrogen at a pressure of about 0.6 mm. of mercury. The light from the water spark was directed by the lens L through the glowing hydrogen to the slit H of the spectrograph. The Balmer lines appeared as fine dark absorption lines against the bright background spectrum. On the original plates the series extended clearly to H_{10}; the reproduction given in a, Fig. 10, is not very

[24] *Amer. Phys. Soc. Bull.*, Feb. 13, 1926.
[25] HULBURT, *Phys. Rev.*, 23, 593 (1924).

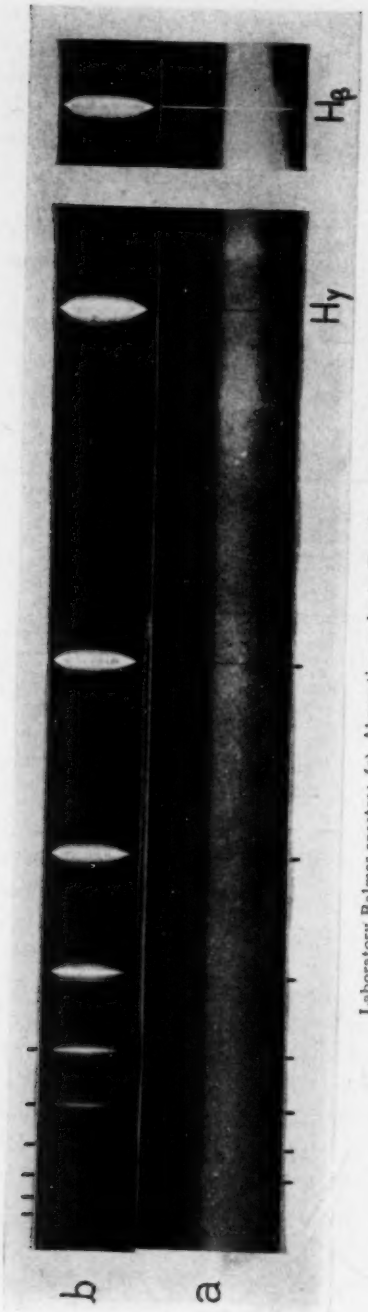


FIG. 10.

Laboratory Balmer spectra: (*a*) Absorption series to H_{10}; (*b*) emission series to H_{12}.

good. (Spectrogram *b* (Fig. 10) is a comparison emission spectrum taken with another hydrogen tube.) It is worthy of mention that the apparatus of Fig. 9 is the best imitation of a Huggins star yet made in the laboratory.

The time of exposure for spectrogram *a* was five minutes. In the emission spectrum of the tube, under exactly the same conditions as for *a*, except with the water spark removed, with an exposure of fifteen minutes, the series lines could be traced only to H_6. This emission spectrum is shown, none too well, just above *a*. With a five-minute exposure the emission spectrum barely included H_4. It is evident that under these conditions the absorption brings out many more lines than the emission. Apparently the stimulated atoms are very effective absorbers for the Balmer radiations. This is in keeping with the classical theory of absorption discussed mathematically by Lamb,[26] who pointed out that a single resonator can exert enormous checking power upon advancing radiation of a frequency very nearly, but not quite, that of its own period. A similar instance is the high opacity of mercury vapor for the mercury line 2536.[27]

One would suppose that Whiddington's experiment could be adapted to the production of the Balmer absorption lines. An attempt to do this, however, met with failure.[28] And, although we really do not do justice to the experiment in such brief mention, it would seem that perhaps the pressures of hydrogen used, 0.1 to 5 mm., were not low enough.

The reason that the absorption series is fully developed in many of the stellar spectra and the emission in so few follows at once. To present a many-lined Balmer absorption series requires a bright interior emission from the star through a relatively cool hydrogen envelope. This may be regarded as a normal condition and one likely to remain stable for some time. In order to emit an extended series of bright Balmer lines, each line being simple, such as those of spectrogram *Bp* (Fig. 2), the star must possess a radiating atmosphere of intensely excited hydrogen, with no cooler absorbing layers above. Such a condition is unstable, as has been pointed out in an earlier paragraph, and

[26] LAMB, *Cambridge Phil. Soc. Trans.*, **18** (1900).
[27] WOOD, "Researches in Physical Optics," Part I.
[28] JOLOWICZ, *Phil. Mag.*, **50**, 1089 (1925).

would not be expected to persist. It might arise when the star explodes with great flames of hydrogen. After such a catastrophy the expelled hydrogen gas would eventually surround the star in the form of a cool envelope, and the Balmer lines, previously bright, would appear as dark reversals. Exactly these changes have been observed in certain stellar spectra. In 1888 H_α, H_β, and H_γ of *Pleione* and of μ *Centauri* were very bright.[3] During the years around 1905 the lines became weaker and after passing through various complex changes now survive only as dark absorption lines.

COMPLEX BALMER LINES.

Referring to the absorption experiment just described, it is seen that all the Balmer lines of spectrogram *a* (Fig. 10) were reversed except H_β, which was bright. A visual observation at the time showed that H_α also was bright against the continuous background. Decreasing the brilliancy of the glowing hydrogen by admitting a small amount of hydrogen into the tube, brought out H_β dark instead of bright against the continuous spectrum. Increasing the brightness of the glowing hydrogen gave a spectrum with H_α, H_β, and H_γ appearing as emission lines and the other lines as absorption lines. The explanation of these observations is to be found in the conclusions of Ladenburg [29] as a result of an investigation of the absorption of H_α. By means of nicol prisms the relative brightness of H_α and the background illumination were varied at will. It was established that in the region of wave-lengths covered by the Balmer line the light from the background was completely absorbed, and that the light which did appear in this region was emitted from the column of luminous hydrogen. Therefore, the appearance of the Balmer lines as bright or dark depended primarily upon the relative brightness of the two sources.

Interesting counterparts of these phenomena are found in the stellar spectra, some stars [30] as γ *Argus* and η *Tauri* having the earlier members of the Balmer series bright and the shorter wave-length hydrogen lines dark. One may suppose that in these cases the hydrogen envelope was sufficiently cool (*i.e.*, unexcited) compared with the hot interior to darken the shorter lines and yet sufficiently hot (*i.e.*, excited) to give the longer lines brighter

[29] LADENBURG, *Verh. Deut. Phys. Ges.*, **12**, 54 and 549 (1910).

[30] CAMPBELL, *Astrophys. J.*, **2**, 177 (1895).

than the background spectrum. Questions of the distribution of intensity in the Balmer series and in the continuous spectrum enter here, but their consideration must be left to the future.

In the absorption experiment spectrograms were also obtained in which H_α and H_β were bright, H_γ was dark, with a bright centre, and the other lines were entirely dark. Other spectrograms under slightly different conditions showed H_α bright, H_γ dark with a bright centre, and the remaining lines dark. That the dark absorption line may have a bright centre was observed by Ladenburg,[28] who proved that the dark edges of the line were due to a complete absorption of the background spectrum by the long layer of glowing hydrogen and but weak emission by the hydrogen itself, whereas in the centre the emission of the hydrogen was strong. Again these phenomena are well known in the stars. For instance, the quasi-permanent stars *μ Centauri* and *J Velorum*[3] at one time exhibited H_β and H_γ as absorption lines with well-marked emission centres. In the spectrum of *II Camelopardalis* given in spectrogram 2 (Fig. 3), all the lines appear to be dark with bright centres of emission. It must be emphasized that the bright centres are not radiations of the interior peeping through small spectral gaps in the surrounding hydrogen envelope. The conditions in the star are rather to be pictured as follows: (*a*) A hot interior giving a continuous spectrum, (*b*) surrounded by an absorbing envelope, relatively dense (or suffused with electrons and ions) giving a wide absorption line, (*c*) which in turn is surrounded by hydrogen at less pressure and greater excitation, thereby emitting a narrower line.

A further complexity of certain stellar hydrogen lines which as yet has not been imitated in the laboratory is to be found in the triply reversed lines, as, for example, H_γ, H_δ and H_ε in the spectrum of *φ Persei* and H_β and H_γ in *β Monocerotis*. The spectra are given in Fig. 2. These lines are broad, dark absorption bands with a narrow, bright centre band, which in turn is divided by a yet narrower line of absorption. A star which would emit such a complex line would be like the star described in the preceding paragraph, enveloped, however, by a fourth region of cooler, more rarified hydrogen which would cause the narrow centre of absorption.

Finally, we must call attention to other peculiarities of stellar

hydrogen lines [31, 32, 33, 34] observed for the most part in the variable stars, which as yet have no adequate explanation. The hydrogen lines often change in intensity, each in a characteristic manner, with the variation in stellar magnitude of the star. Unsymmetrical double and triple reversals sometimes occur. For example, the hydrogen lines from H_β to H_5 in the spectrum of H.D. 108 consist of rather narrow bright lines superimposed on faint dark lines, the dark edges of shorter wave-length being slightly more intense than those of longer wave-length.[32] Of particular interest are the spectra of the *P Cygni* type; the spectrum of *P Cygni* appears in Fig. 2. Here the Balmer lines are bright with dark components on the violet side. Although the origin of such lines is quite obscure, one wonders whether it is necessary to invoke a "veil theory" [35] in explanation. Perhaps present knowledge of the hydrogen spectrum is insufficient. Indeed, in the water spark spectra [36] of iron, copper, gold, platinum, rhodium, and perhaps other metals, there are many bright metallic lines with dark components to the short wave side just as the hydrogen lines of *P Cygni*. As far as I know, a bright line with a dark component on the red side occurs in neither the water spark nor the stellar spectra. It would appear not impossible, although perhaps improbable in view of the symmetrical nature of the Stark and Zeeman patterns of the Balmer lines, to discover ways of producing the lines unsymmetrically broadened and reversed, perhaps even apart from Doppler effects.

ERRATUM.

SPACE CHARACTERISTICS OF ANTENNÆ.

BY

William H. Murphy,

Captain, Signal Corps, U.S.A.

Formula (8), page 418 (April, 1926), should read:

$$\dot{E} = \sqrt{\sin^2\beta\cos^2\theta_1 + \sin^2\theta_1}\,.$$

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[31] LOCKYER, *Roy. Ast. Soc.*, Monthly Notices, **84**, 409 (1923–1924).

[32] MAURY, *Harvard College Observatory. Annals*, **29**, 105 (1897); *et al.*

[33] MERRILL, *Lick Observatory Bull.*, **7**, 162 (1913); *Astrophys. J.*, **53**, 1 (1921); **61**, 1 (1925).

[34] Also see *Roy. Ast. Soc.*, Monthly Notices, **84**, 272 (1923–1924).

[35] MERRILL, *Astronomical Observatory Pub., Univ. of Mich.*, **2**, 70 (1916).

[36] HULBURT, *Phys. Rev.*, **24**, 129 (1924).

NOTES FROM THE NELA RESEARCH LABORATORY.*

COLLECTOR CHARACTERISTICS IN HELIUM.

By Richard Rudy.

THE primary electrons reaching a collector which is negatively charged with respect to the anode were studied at different pressures of helium (.01 to .1 mm.) in the same way as for neon, the discharge current being 5 m-amp. and the anode being at + 70 volts. As the distance of the collector from the cathode was increased, the number of electrons collected decreased, owing to collisions with the gas molecules, and at the same time the slope of the collector volt-ampere characteristic as far as it was determined by primary electrons decreased, showing that even electrons which passed through the gas without colliding lost energy. For the same pressure and the same distances, the loss of energy was several times smaller than in neon. It could not be determined as accurately as in neon, because with the type of discharge tube used, the different characteristics were not well enough separated at lower pressures, and at higher pressures the straight portion was too short. Theory requires a loss about four times smaller than in neon, which is of the order observed. When, in neon, the distance of the collector was increased, the number of the electrons arriving against the retarding field decreased exponentially; in helium it was more difficult to verify the law because even up to .04 mm. mercury the collector current decreased after having reached the saturation value.

When the current was raised to about 10 m-amp., the break from the positive saturation current to the electron current took place at voltages against which the 70-volt electrons could not travel. As in neon, this type of characteristic was obtained after a sudden discontinuity had taken place in both types of discharge tubes used, one having a disc, the other a cylindrical anode. The exact value of the current where the break took place seemed to depend somewhat on charges upon the wall or the stems. This jump was usually small, often a few micro-amps., and a change in luminosity could not be expected. But in some cases, especially with the collector in the positions close to the anode, it

* Communicated by the Director.

amounted to ¾ m-amp. In this case, and favored by the bluish-green color of the discharge in helium, a sudden change in the distribution of luminosity was detected when a region protected from the bright light from the filament (*e.g.*, between stem and wall) was examined.

The discharge was blue at lower pressures and turned to green when the pressure was increased.

ON THE STABILITY OF GLOW DISCHARGES.

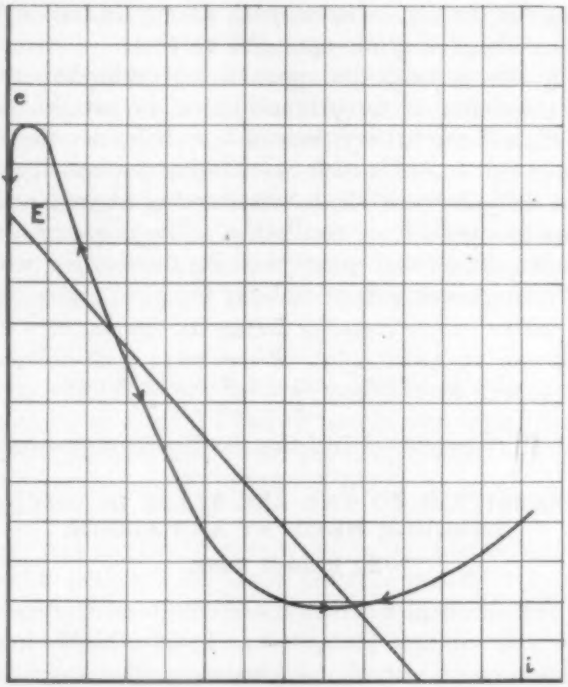
By Richard Rudy.

THE volt-ampere characteristic of a glow discharge begins at a voltage near the starting potential, V, falls to a flat minimum, which for short distances is not far from the normal cathode fall, and then rises again, owing to the cathode fall becoming abnormal. For the simple circuit, including resistance, R, inductance, L, and a constant e.m.f., all in series, the straight line from E [1] having as its slope $-1/R$ intersects the characteristic curve at the points where a discharge is possible; but the discharge is continuous and stable only where the slope of the characteristic curve is steeper than the resistance line; the equation of the circuit for a very small change η_0, impressed upon the circuit is $\eta = \eta_0 E^{-\frac{1}{L}\left(R + \frac{\delta e}{\delta i}\right)}$. The ionization going on in the tube is therefore responsible for oscillations only as far as it influences the shape of the characteristic. This equation is sufficient only for small changes, Δ_i, produced. As soon as the variation goes beyond the next unstable point of intersection, the discharge may stop and the figure shows at once that $\frac{\Delta e}{\Delta i} + R$ must stay positive. If this is not the case, there is a flashing discharge. For the maintenance of a discharge, a certain flow of electricity is indeed necessary, and if an effort is made to stabilize a discharge by increasing the external resistance, a point is reached where the quantity of electricity arriving through the resistance upon the electrodes is no longer sufficient for the discharge, the discharge stops, and only after sufficient electricity has arrived is a new flash possible. In other words, the capacity of the electrodes is no longer negligible, and one is led to distinguish between the

[1] E may be smaller than V because we may start the discharge with a transformer.

charging current (maxwellian displacement current) and the discharge current. It is common practice to increase the capacity of the electrodes by putting a condenser in parallel with the electrodes or with the battery. There is then a critical value of the resistance, R_c, below which no flashing is possible, *i.e.*, when $\frac{E-e}{R} = i$. It may be seen from Fig. 1 that the different formulas

FIG. 1.



deduced for R_c express nothing but the above conditions, *i.e.*, that $\frac{\Delta e}{\Delta i} + R$ must be positive. A self-inductance is of course not required for these "oscillations."

The presence of capacity allows the circuit to oscillate; and a new condition must now be satisfied if the discharge is to be continuous, *i.e.*, stable with regard to oscillations. Not only must $de/di + W > 0$, but $de/di + L/CR > 0$. When de/di is positive, both conditions are always fulfilled; but, for a falling characteristic, it may be seen again that one cannot always render a discharge

stable and continuous by merely increasing the resistance. Oscillations may be produced or stabilized by increasing R too much.[2]

Owing to hysteresis, a discharge presents always a certain self-induction, and the voltage variations lag behind the current variations. If a discharge is completely free from hysteresis, as happens with electron tubes, then all the conditions concerning stability have their sign reversed. The reason is that on account of the capacity, the potential is in advance of the current and determines the current, whereas with strong ionization in a discharge the voltage depends upon the current.

Taking into account the apparent self-induction, the circuit then really consists of three branches, all in parallel: Branch a with L_0, R_0 and the battery, branch b with R, the discharge and its self-induction L, and branch c containing the capacity C. Oscillations are therefore possible in two branches of the circuit, or the system has two periods of oscillation. The discharge current is found from a differential equation of the third order, which leads to the following conditions of stability:

$$(L + L_0) + CRR_0 > 0$$
$$R + R_0 > 0$$
$$(L + L_0 + CRR_0)\ (LR_0 + L_0R) > (R + R_0)\ LL_0$$
$$LR_0 + L_0R > 0$$

Here R or R_0 represents or includes the discharge resistance de/di.

THE TRANSITION TO THE ARC STAGE IN DISCHARGES THROUGH MERCURY AND ARGON.

By Richard Rudy.

IT IS still doubtful whether the arc requires for its existence an incandescent cathode (tungsten arcs) or a highly ionized gas (alkali vapor arcs), or both, or whether another agent besides is responsible for the low cathode fall. Possibly, more work on the transition between different stages of the discharge helps settle the question. Attempts were made with mercury vapor to pass from the discharge with the incandescent filament cathode to the arc discharge (self-sustained discharge), the tube being heated in an electric furnace. The distance between the electrodes was from ½ to 2 cm. Once the saturation current passed, the current increased steadily as the applied voltage increased. With the

[2] As a possible illustration of this case, see Eckart and Compton, *Phys. Rev.*, **24**, 103 (1925).

filament as hot as it could stand without melting and with discharge currents of nearly 1 amp., the discharge always stopped as soon as the heating current was interrupted. The filament required about 80 watts at 12 amp., and 180 watts at 16 amp. The mercury-vapor pressure was of the order of 1 or 2 cm.

As soon as argon, at a pressure of 2 mm., was added, two distinct families of characteristics were observed. For the filament currents below 12 amp. the characteristic was entirely altered from what it had been with mercury alone; above the saturation value the current rose first with increasing voltage, but soon a point was reached where the voltage remained constant and then began to fall to lower values while the current increased. This falling characteristic, having the shape of a hyperbola, could be shifted at will towards higher voltages, without its shape being changed, by decreasing the heating current from, for example, 12 amp. to 6 amp. or less. The amount of heat given to the filament was not, however, entirely used up to lower the voltage, so to speak, and a complete transformation would, of course, be impossible. An increase of about 10 watts in the heating energy, S, corresponded to a reduction in the ei value of the discharge of 2.5 watts. The difficulty in these experiments in studying this interesting point in Simon's theory of the arc, which has never been tested quantitatively, was the evaporation of mercury and the unknown length of the filament actually supporting the discharge.

Contrary to what might be expected, when a current of .5 amp. was allowed to pass and the argon pressure was then increased to 1 cm., the same current was now again obtained at lower voltage, a reduction to almost half the value being noted in certain points. Slight reductions took place up to an argon pressure of 5 cm., the highest tried. When the filament current was decreased, by steps of 1 amp., displacements of the characteristics were observed as in the case of argon at 2 mm., but they were smaller.

Outside of this influence upon the shape of the characteristic, the presence of argon now allowed the filament current to be interrupted without the discharge being stopped. A self-sustained discharge was then obtained at a low temperature of the filament (equivalent to a heating current of a few amperes). The shape of the volt-ampere plot remained unchanged.

Besides this group of characteristics which shows the influence

of increased electron emission upon the cathode fall, a second type was observed for filament currents over 12 amp. Instead of falling, when the current increased, the voltage remained almost perfectly constant or rose very slightly. On still further increasing the filament temperature, the readings of the voltage for a given discharge current were lower, but the difference was only about ½ volt when the filament current was made 16 amp. instead of 15 amp. With the higher filament temperatures the discharge current was evidently maintained to a large part by the thermionic emission; the equilibrium between positive and negative charges was entirely different and ionziation was produced in fields which were different from those where the falling characteristics were obtained. They are similar to the volt-ampere plots with mercury alone present.

The highest electron emission for which a falling characteristic could be noted was 250 m-amp. for 12 amp. heating current, where sometimes transitions between both types took place.

Variation of the electrode distance was of little influence upon the results. The discharge had the blue color of the mercury discharge. It was of course difficult to know whether argon lines were entirely absent in the immediate neighborhood of the bright filament, and it is just in this narrow zone of the cathode fall where argon exerts its influence by reducing the rate of diffusion or by increasing the number of metastable atoms.

The experiments were carried out in three different discharge tubes with the same result. As soon as the argon was removed the falling characteristic disappeared. The work was interrupted some time ago, but it is intended to take it up again with argon alone present.

Dust Explosions.—Practically all combustible dusts will explode. The right conditions are a dust-laden atmosphere, oxygen and a means of ignition. The rapid combustion of the fine particles of dust results in an explosion of terrific force. The seriousness of the problem and the need for expending every effort in applying known control measures may be realized from a glance at the figures gathered by the Department of Agriculture. Twenty-eight thousand plants in the United States are subject to the hazard. These plants employ 1,324,422 people whose lives would be endangered, and manufacture products worth more than $10,000,000,000. Two hundred and eighty-one explosions are on record. In 70 cases, 459 people were killed. In 92 cases, 760 people were injured. The average property loss for each explosion is estimated at $240,000. H. L.

NOTES FROM THE U. S. BUREAU OF CHEMISTRY.*

A COMPARATIVE STUDY OF THE CHEMICAL COMPOSITION OF THE SARDINE (SARDINEA CÆRULEA) FROM CALIFORNIA AND BRITISH COLUMBIA.[1]

By D. B. Dill.

[ABSTRACT.]

AN investigation to determine whether or not significant chemical differences occur in the sardine (*Sardinea cærulea*) at two widely separated areas in its range was conducted in the Bureau of Chemistry. The following results were obtained:

Two samples of British Columbia sardines showed a higher fat content than has ever been found in California sardines.

A representative sample of sardines from British Columbia agreed strikingly in composition of ash and in characteristics of oil with a representative sample of sardines from California.

No evidence of seasonal variations in the constants of the oil of the California sardines was found.

In the process of sterilization of sound sardines in a sealed container, no notable hydrolysis of the fat was evident.

THE REFRACTOMETRIC DETERMINATION OF ALCOHOLS AND ESTERS IN AQUEOUS AND IN COTTONSEED OIL SOLUTIONS.[2]

By James C. Munch.

[ABSTRACT.]

A METHOD to determine the partition coefficients between oil and water for a number of aliphatic alcohols and alkyl esters, has been devised in the Bureau of Chemistry. The refractive increments of these alcohols and esters in aqueous and in cottonseed oil solutions were determined. Quantitative determination of the products may be readily and accurately made by the use of these increments. There is significant correlation between the refractive increments of aqueous and of oil solutions: ($r=-0.78\pm 0.04$). Thus it becomes possible to calculate the increment for either solution if the value for the other is known.

* Communicated by the Chief of the Bureau.

[1] Published in *Ecology*, 7, No. 2 (April, 1926), 221, 228.

[2] Published in the *J. Am. Chem. Soc.*, **48**, No. 4 (April, 1026), 994–1003.

CHANGES IN NITROGEN, POTASSIUM, AND PHOSPHORUS CONTENT OF WHEAT SEEDLINGS DURING GERMINATION AND EARLY STAGES.[1]

By Jehiel Davidson.

[ABSTRACT.]

THE Bureau of Chemistry has completed an investigation to determine experimentally whether seedlings, at the ages at which they are generally transferred to water or sand cultures, differ in composition from the stock seed from which they originate. The results showed that they do. The experimental seedlings either lost or gained potassium and nitrogen, depending upon their age and their conditions of growth. They changed little in phosphorus content, however.

The Variation of the Thermal Conductivity of Gases with Pressure. H. GREGORY and C. T. ARCHER. (*Phil. Mag.*, March, 1926.)—From the kinetic theory of gases Maxwell drew the conclusion that the thermal conductivity should not change when the pressure of the gas changes. The transfer of heat from one surface limiting a portion of gas to another such surface at a lower temperature is effected by the migration of molecules of greater kinetic energy toward the cooler surface. This is in part compensated by the motion of molecules with less energy in the opposite direction. On the whole, more energy is carried toward the cooler surface than toward the warmer one. When the pressure is reduced there are fewer molecules for the transport but, on the other hand, they have longer free paths. These two opposing effects neutralize each other so that the thermal conductivity remains unchanged when pressure varies. This invariability of conductivity has been previously shown to exist by several investigators, but the improvement in methods of measurement made it desirable to examine the matter anew with the application of modern gains in accuracy. This the authors of the present paper have done. They find no departure from constancy to be indicated by their method; the range of pressures was from 76 cm. to about 6 cm. of mercury. A marked reduction in conductivity showed itself when the length of the free mean path of the molecules became comparable with the distance from the warmer to the cooler surface. This change is quite in agreement with theory and in no way invalidates the general conclusion of the constancy of conductivity with reduction of pressure. G. F. S.

[1] Published in the *Botanical Gazette*, 81, No. 1 (March, 1926), 87–94.

THE FRANKLIN INSTITUTE.

(Proceedings of the Stated Meeting, Wednesday, May 12, 1926.)

THE stated monthly meeting of the Institute, at which the exercises in celebration of Medal Day were held, was called to order at three-thirty-five by President Wm. C. L. Eglin.

The President stated that the minutes of the April meeting had been printed in full in the May number of the JOURNAL, and that, unless there were objection, they would be approved as printed. No objection was offered, and the minutes were declared approved as printed.

The Secretary announced that at the latest meeting of the Board of Managers, eight new members had been elected to the Institute.

The Secretary announced, with sincere regret, the deaths of four members of the Institute since the last meeting, namely, Messrs. G. Aertsen, V. Angerer, C. Hering, and S. P. Wetherill.

The Secretary made announcement of special exercises of the Institute to be held on Friday afternoon, May twenty-first, at four o'clock, at which time formal presentation to the Institute will be made of a portrait of Dr. Samuel Insull, of Chicago, President of the Commonwealth Edison Company, and donor of the Franklin Medal Fund. He invited all members of the Institute and their friends to be present at these exercises.

The exercises in connection with the presentation of the various Medals were then held.

PROGRAMME

	Presented by:
Presentation of the Longstreth Medal to	
MR. JAMES JOHN DENNY	Mr. H. H. Quimby
MacIntyre Porcupine Mines, Ltd.	
Awarded in absentia	
Received by The British Consul General	
Presentation of the Wetherill Medals to	
MR. FRANK TWYMAN, F.R.S., F. INST. P.	Dr. J. Barnes
Managing Director, Adam Hilger, Ltd.	
Awarded in absentia	
Received by The British Consul General	
THE WAGNER ELECTRIC CORPORATION	Mr. Harold Calvert
Mr. E. W. Goldschmidt	
In person	
Presentation of Levy Medal to	
MR. FRANK W. PEEK, JR.	Dr. H. J. Creighton
General Electric Company	
In person	

	Presented by:
Presentation of Levy Medal to PROFESSOR ERNEST G. COKER, D.Sc., F.R.S.E. University of London Awarded in absentia Received by The British Consul General	Dr. Creighton
Presentation of Cresson Medals to GEORGE ELLERY HALE, PH.D., D.Sc., LL.D. Honorary Director, Mount Wilson Observatory Awarded in absentia	Prof. W. O. Sawtelle
CHARLES SHELDON HASTINGS, PH.D. Professor Emeritus of Physics, Yale University In person	Dr. G. S. Crampton
Presentation of the Franklin Medal and Certificate of Honorary Membership to PROFESSOR NIELS BOHR, PH.D. Director of the Institute for Theoretical Physics, Copenhagen, Denmark Received by the Minister of Denmark to the United States	Prof. W. F. G. Swann
Presentation of the Franklin Medal and Certificate of Honorary Membership to SAMUEL REA, Sc.D., LL.D. Former President, Pennsylvania Railroad Received in person	Dr. Walton Clark, read by Mr. M. S. Morgan

Paper: "Engineering Reminiscences," by SAMUEL REA, Sc.D., LL.D.
Adjourned.

HOWARD MCCLENAHAN,
Secretary.

(An extended account of the proceedings of this Medal Day meeting will appear in a later issue of the JOURNAL.)

COMMITTEE ON SCIENCE AND THE ARTS.

(Abstract of Proceedings of Stated Meeting held Wednesday, May 5, 1926.)

HALL OF THE COMMITTEE,
PHILADELPHIA, May 5, 1926.

MR. CLARENCE A. HALL, *Chairman.*

The following report was presented for final action:

No. 2815: Conversion of Acetylene to Acetaldehyde and Its Oxidation to Acetic Acid.

This is a process for the production of acetic acid from acetylene in commercial quantities, which was developed to meet a war-time demand for the acid. Manufacturing on a laboratory scale was developed in 1915 and the method was perfected to such a degree that commercial production was begun in December, 1916, millions of pounds of acetic acid being produced during 1917.

The process includes three steps:

(1) The manufacture of acetaldehyde from acetylene.

(2) The manufacture of mercury compounds from mercury.

(3) The manufacture of acetic acid from acetaldehyde.

The fundamental reactions for steps 1 and 3 were well known and patents upon them had been granted but no process that would give amounts of the products in commercial quantities had been developed prior to December, 1916.

In general, the process consists in the generation of large quantities of acetylene from calcium carbide; the absorption of the acetylene in a warm 6 per cent. sulphuric acid solution with a mercury salt as a catalyst to form acetaldehyde; the distillation and purification of the aldehyde; the oxidation of the aldehyde by air in aluminum kettles at a relatively low temperature and under a pressure of several atmospheres, in the presence of a catalyst and distillation of the resulting acetic acid giving a 99–99.5 per cent. glacial acetic acid.

This report, recommending the award of the Howard N. Potts Medal to Mr. Howard W. Matheson, of Shawinigan Falls, Canada, "in consideration of the inventive skill and ingenuity and the application of scientific information displayed in the development of a successful commercial process for the oxidation of acetylene to acetaldehyde and acetic acid from previously known chemical reactions," was adopted.

The following reports were presented for first reading:

No. 2853: The Coolidge Tube.

No. 2855: Fuller-Kinyon Conveying System for Pulverized Materials.

GEO. A. HOADLEY,
Secretary to Committee.

MEMBERSHIP NOTES.

ELECTIONS TO MEMBERSHIP.

(Stated Meeting, Board of Managers, May 12, 1926.)

RESIDENT.

MR. R. B. ERICKSON, Metallurgist, Hancock and Berk Streets. For mailing: 4620 North Camac Street, Philadelphia, Penna.

DR. GUSTAVE E. LANDT, Chemist, Diamond State Fibre Company, Bridgeport, Penna.

DR. JAMES DOUGLAS MORGAN, Physician, 2226 Delancey Street, Philadelphia, Penna.

MR. GEORGE J. SIEDLER, Sales Engineer, Haverford, Penna.

MR. ROBERT STINSON, Chemist, 68 East Penn Street, Norristown, Penna.

MR. GEORGE W. PRESSELL, Chief Chemist, E. F. Houghton and Company, 240 West Somerset Street, Philadelphia, Penna.

NON-RESIDENT.

MR. EDWIN H. BURK, Merchant, 2118 Atlantic Avenue, Atlantic City, N. J. For mailing: 105 South Derby Avenue, Ventnor, N. J.

MR. ALBERT WOOD MORRIS, Consulting Engineer, 701 Morris Building, Philadelphia, Penna. For mailing: 54 Buckingham Street, Springfield, Mass.

CHANGES OF ADDRESS.

MR. J. SNOWDEN BELL, Hudson Terminal Building, Rooms 1422–1424, 30 Church Street, New York City, N. Y.

MR. R. W. A. BREWER, East Township Line, Jenkintown, Penna.

MR. G. E. CHAMBERLAIN, 446 North Oak Crest Street, Decatur, Ill.

DR. ALLERTON S. CUSHMAN, 898 Park Avenue, New York City, N. Y.

MR. W. L. DEMPSEY, 400 Granite Building, St. Louis, Mo.

MR. H. H. FURNESS, JR., 2034 Delancey Place, Philadelphia, Penna.

DR. HENRY PHELPS GAGE, 214 Chemung Street, Corning, N. Y.

MR. A. H. GFRORER, 2933 Main Street, Buffalo, N. Y. Chief Engineer, The Automatic Transportation Co.

MR. EDWARD HEITMAN, 790 Broad Street, Newark, N. J.

MR. W. N. JENNINGS, 2947 South Broad Street, Philadelphia, Penna.

MR. LEGRAND PARISH, Mountain View, N. J.

MR. MONROE B. SNYDER, 120 Woodside Avenue, Narberth, Penna.

MR. C. F. SULZNER, 236 South West Second Street, Miami, Fla.

MR. C. H. UMSTEAD, In care of—General Delivery, Portsmouth, N. H.

MR. WALTER C. WAGNER, 2301 Market Street, Philadelphia, Penna

MR. FRANCIS H. WYETH, 6331 Magnolia Avenue, Germantown, Philadelphia, Penna.

NECROLOGY.

Mr. Guilliaem Aertsen, Philadelphia, Penna.
Mr. Victor Angerer, Ridley Park, Penna.
Dr. Carl Hering, Philadelphia, Penna.
Mr. Samuel P. Wetherill, Philadelphia, Penna.

LIBRARY NOTES.

RECENT ADDITIONS.

American Society for Testing Materials.—Proceedings: Twenty-eighth Annual Meeting. Vol. 25, Parts 1 and 2. 1925.

BOLTON, D. J.—Electrical Measuring Instruments and Supply Meters. 1923.

CHAMBERLAIN, JAMES FRANKLIN.—Geography: Physical, Economic, Regional. 1921.

Chemiker-Kalendar, 1926. Three volumes.

Chemisches Zentralblatt.—Generalregister VI über die Jahrgänge 1922–1924. Teil I Autorenregister. 1925.

DUNOYER, L.—La Technique du Vide. 1924.

Engineering Index, 1925. 1926.

FIELD, PETER.—Projective Geometry with Applications to Engineering. 1923.

HADFIELD, SIR ROBERT A.—Metallurgy and Its Influence on Modern Progress, with a Survey of Education and Research.

HINZE, ADOLF.—Die Weisszucker-Herstellung in den Rübenzuckerfabriken. 1925.

LEBEDEW, PETER.—Pression de la Lumière. Traduit du Russe par T. Kousmine. 1926.

MILLER, EDITH M., and Others.—Some Great Commodities. 1922.

NODON, ALBERT.—Éléments d'Astrophysique. 1926.

SIEMENS-KONZERN.—Wissenschaftliche Veröffentlichungen. Vol. I, Parts 2–3, Vols. II–IV. 1921–1925.

WENTZEL, FRIEDRICH.—Die photographisch-chemische Industrie. 1926.

BOOK REVIEWS.

HANDBUCH DER PHYSIK. A series of twenty-four volumes prepared under the editorial coöperation of Messrs. Henning, Trendelenberg and Westphal, of Berlin; Thirring, of Vienna; Konen, of Bonn, and Grammel, of Stuttgart, assisted by numerous collaborators. Issued by H. Geiger and Karl Scheel. Vol. 10, Thermal Properties of Matter, edited by F. Henning. vii–486 pages, 207 illustrations, 8vo. Berlin, Julius Springer, 1926. Price, 35.40 marks.

From a slip attached to a loose sheet containing the preface to the series, it is learned that this volume is the first to appear. Irregularity in publication is often unavoidable in the issue of an extensive series, as authors do not by any means all "come to time," and as they say of convoys that the speed is that of the slowest ship, so the manuscripts promptly furnished have to be held until all are in. The series of which this large and well-printed volume is a member is a very extensive one, perhaps the most extensive yet initiated in the field of physics. It is, indeed, astonishing that even Germany should be able to take such a book in quantity sufficient to repay the outlay, for although it will find its way into the libraries of scientists of other nations, such distribution cannot be very extensive. It is well worth a place in the libraries of all engaged in teaching physics or in research in physical or physico-chemical lines.

The series is subdivided into groups as follows: Volumes 1 to 3 include history, units, mathematical aids; 4, fundamentals; 5 to 8, mechanics, including acoustics; 9 to 11, heat; 12 to 17, electricity and magnetism; 18 to 21, optics of all wave-lengths; 22 to 24, the nature of matter and the phenomena of radiation. An extensive division of the whole field, and distribution of the several topics to specialists ensures a thorough and up-to-date presentation of data, though it imposes a considerable burden upon the editors, who are likely to find frequent overlaps, and, therefore, conflicts of view between writers.

The volume in hand will be seen by the above enumeration of assignments to be part of the section on heat. It deals with the thermic properties of matter and includes: Conditions of solid fusion, freezing and sublimation, conditions of gases and liquids, thermodynamics of mixtures, specific heat—theoretical and practical; determination of free energy; thermodynamics of solutions. Each of these subjects is the work of a special contributor.

Space does not permit of an extensive review of this comprehensive and ably written volume. The several topics are exhaustively treated and the numerous illustrations add to the clearness and usefulness of the text. Mathematical demonstrations are extensively used. In the first section a somewhat novel treatment of the periodic table is presented. The crystalline form of many of the elements is given and the study is summarized by the statement that thirty-two crystallize in the isometric system, eleven in the hexagonal. The systems of minor symmetry are represented by comparatively few. It must be noted, however, that the sum of those assuming the regular system is less than half the number of known elements, and in interpreting the significance of crystalline form of elements regard should be given to the fact that the great bulk of them are very similar in physical nature, being included in the group of metals, although it is true that this group cannot be very closely defined. Some common elements, *e.g.*, sulphur and carbon, exist in two very distinct forms, and several of those gaseous at even low temperatures have not yet been examined as to form.

The relation between crystalline form and chemical composition is also discussed. Investigations made in 1846 by Buys-Ballot led to the view that the simpler the molecule, the more likely it is to appear in a symmetric form. Investigations by J. W. Retgers in 1894, which covered about one thousand inorganic compounds, gave noticeable percentages of symmetric forms in substances containing few atoms. Over five hundred organic compounds are also tabulated, but these are not classified by numbers of elements present. It is a question, however, whether the number of substances examined is enough to constitute the basis for a law. Peculiar exceptions to the rule occur. Carbon, an element; calcium oxalate, a compound of three elements; alum, a compound of five, crystallize in regular octahedrons. Uranium-sodium acetate, also a compound of five elements, crystallizes in regular tetrahedrons.

Typographic errors are uncommon in German and French books, which seem to be very carefully proof-read, but a curious and somewhat amusing error is noted in the summary of subjects of the several volumes. In the summary for Vol. 21, Professor Dr. J. Hopmann, of Berlin, is announced as contributing a section on "Application to comic questions" (Anwendung auf komische Fragen). Modern physics is no joke, and the omission of the "s" makes a notable change in significance though, of course, not in any way misleading, as the intended reading is obvious. The completion of this "Handbuch" will constitute a most useful addition to the literature of physical science and its related subjects.

HENRY LEFFMANN.

METALLURGY AND ITS INFLUENCE ON MODERN PROGRESS WITH A SURVEY OF EDUCATION AND RESEARCH. By Sir Robert A. Hadfield, Bt., D.Sc., F.R.S. xvi–388 pages, numerous illustrations in the text and many plates. New York, D. Van Nostrand Company, 1926. Price, $7.50.

The title of this work is in one way too comprehensive and in another way not as comprehensive as deserved. It gives one a notion that it covers the general field of metallurgy, whereas it is practically limited to ferrous alloys. On the other hand, it contains a very large amount of general information concerning the history of iron and the making and using of its important alloys.

Iron is indeed the most important metal in our present-day civilization. The story of its introduction into human life is an essential item in comprehending the course of human development over many centuries. The author has a wide knowledge and profound interest in the course of human affairs, and while he enjoys setting forth the past, he also responds fully to the present and looks with keen eyes into the future. We might, indeed, ask ourselves what would have been the history of mankind if the working of iron and the manufacture of steel had not been developed. In view of the terrors of modern warfare in which the means of offense seem to have outstripped the means of defense, the pessimist might say that the introduction of iron has made matters worse.

Notes of the history of metallurgy are given, beginning with Roger Bacon, who from recent discoveries seems to have been a worker of the type of Leonardo de Vinci. A notice is also given of the better known Bacon—Lord Verulam—who is generally claimed as the founder of inductive science. Robert Boyle is duly designated as the "father of modern chemistry" and we are told that he introduced the word "analysis." Discussions of history of science and research labors take a wide range and, as we might expect, Faraday is not forgotten. The Faraday Society, it may be noted, has just celebrated the centenary of the discovery of benzene by that distinguished investigator. Much attention is given to the early history of the steam engine, the prime mover which has had such a powerful and extensive influence in both war and peace. May we say that its reign is near its close? The advancement of water-power, the "white-coal" of the European engineers, and the great vogue of the internal combustion motor may mean, before long, a secondary position for steam. In some respects this is a consummation devoutly to be wished.

A very comprehensive account is given of the application of the microscope to the study of ferrous alloys, a method which is also widely extended now-a-days, being applied to a whole series of materials of construction and of engineering use. Sorby was the pioneer in this line of work, his first applications being to the examination of rock sections and he is justly regarded as the father of petrology. We are told that his suggestions of the value of metallography were received with very hostile criticism, which reminds us of the discouragement that Newlands encountered when he brought before the Chemical Society his suggestions as to the significance of the numerical relations of atomic weights. The Society refused to allow the paper to appear in its *Proceedings*, and one member asked Newlands if he had tried to find any relation between the properties of the atoms and the initials of their names!

The merits of Sir Robert's book are many and great, for it is a most valuable contribution to the field of metallurgy, and as a presentation of the present state of the great industry of modern times. The publisher has done full justice to the author's efforts, the volume being printed in sumptuous form commensurate with the merits of the text.

In a work of so much merit and so abundant in useful information, it is to be regretted that the author has ventured into a discussion of the relations between science and religion. As usual in such allusions, there are merely the glittering generalities of what was termed in former years "natural theology," substantially the deism of the closing years of the eighteenth century, as exemplified in the writings of Paine, Voltaire and Jefferson, differing materially

from the agnosticism of the mid-Victorians. As a matter of fact the "conflict" which is now the occasion of so much book-making and law-making is a conflict of scientific data with the text of the Bible and the two pages which Sir Robert devotes to expressing his views on the relation between science and religion are simply a waste of paper, ink and the compositor's time.

HENRY LEFFMANN.

PROJECTIVE GEOMETRY WITH APPLICATIONS TO ENGINEERING. By Peter Field, Professor of Mathematics, University of Michigan. viii–98 pages, 8 × 5½ inches, illustrations, cloth. New York, D. Van Nostrand Company, 1923. Price, $2.

The theorems of projective geometry are treated at some length in appendices to a few American text-books on geometry where it is styled "modern geometry" and developed without particular reference to its projective origin and, as may be expected in formal text-books on geometry, with no indication of its applications. The present volume has been prepared with the object of bringing within easy reach of American students the substance of the several works of European origin on projective geometry which deal, some with purely mathematical development, others with applications. The inclusion of applications with the formal presentation of abstract matter adds much life and interest to the work, and it should aid materially in stimulating in this country the study of this interesting and useful branch of geometric development.

The book contains an amount of information quite disproportionate to its size. It is much condensed, a habit with mathematical writers who are inclined to a reduction of words to their lowest terms; a slight redundancy of words is often helpful in clarifying a meaning in the briefest possible time.

LUCIEN E. PICOLET.

DRAFT AND CAPACITY OF CHIMNEYS. By J. G. Mingle, C.E. vi–339 pages, 8 × 5 inches, illustrated, cloth. New York, D. Van Nostrand Company, 1925. Price, $3.50.

Of the multiplicity of components which enter into the structure of a power plant, none appears to have received less attention in text or treatise form than the elements of chimney design. The work in hand is written to meet this need of a rational treatise on determining the proper size and the performance of the steam power-plant chimney. The subject-matter originally appeared in serial form in the journal "Combustion."

As relates to chimney draft, the term "draft" by common usage among technical writers means the static force which produces the motion of the column of gases. The use of that term by some writers to designate air in motion is scarcely "unaccountable," as stated, since that is a dictionary definition of the term and is moreover the meaning generally ascribed to it by widespread usage.

The subject-matter is presented in logical order. First an introduction in which a general survey of the subject is made. Then the theory of natural draft is discussed at length in terms of pressure, density and temperature relationships. Under "losses due to velocity and friction," formulas for determining these sources of loss of head are derived at length in terms of

both the volume and the weight of chimney gases. "Available draft," which is the theoretical draft minus the draft lost by the friction and velocity in the stack, is then derived from what precedes. Having determined the available draft, next in its proper order the "general draft equation," by which the losses of head from ash-pit to chimney base are compared, receives attention. This material occupies about one-half of the book. The other topics, which are discussed with equal fulness, are "height of chimney," "elements of combustion," "theory of capacity"—(discharge capacity), "diameter or area of chimney," "altitude and its effect," "chimney performance," "relation between height and diameter," and "economical velocity." An additional somewhat digressive chapter on the "chimney horse-power fallacy" of considerable length is included to stress the objections to this short-cut.

Throughout the work the formulas used are developed in meticulous detail and with each topic a numerical example is fully worked out. There are numerous labor-saving tables and charts in the text. The latter in the main are well executed, but their legends and figures cannot be comfortably read without the aid of a magnifying-glass. A classified key to the symbols used and a more copious index would be helpful.

The work, written in clear and direct language with the formulas employed derived in detail, will appeal alike to the practical engineer and to the engineering student who seek information on chimney draft. To both classes of readers these detailed explanations save time and mental effort.

LUCIEN E. PICOLET.

LES MOTEURS À EXPLOSION. Par Edmond Marcotte, Ingénieur Conseil (I.C.F.), Professeur à l'École spéciale des Travaux Publics, Ancien Chef du Service des Ateliers de Moteurs de l'Aviation Militaire. vii–216 pages, 7 × 4½ inches, illustrated, paper. Paris, Librairie Armand Colin, 1926. Price, 7 francs; bound, 8 f. 50.

A new book dealing with such a well-trodden field as that of the explosion motor must be issued with some special object other than purely technical exposition. The object here is well defined. The work is No. 70 of a series of compendia on a wide variety of technical subjects in a convenient form for ready reference, yet quite comprehensive in character. The latter specification is very adequately fulfilled in its special field of automotive and other motors of relatively small dimensions. Each component of the motor is analytically described with enough formulated data to insure precision. By way of illustration of its fulness of detail, may be mentioned the clear explanation of the diminution of powder of an explosion motor with an increase of altitude, the method of overcoming this obstacle by the use of the "supercharger" and various other matters of interest relating to conditions of navigation at high altitudes. Again, a section on balancing, if summary, is precise and illuminating. The subject-matter is carefully arranged and with its adequate table of contents the work is a very satisfactory one for reference purposes. Although there is no denying the comfort of a complete index, and it must be complete to be of value; it is a question whether an index to a book arranged like this one would add materially to its convenience to the user.

In these days when the cost of books has kept pace with the high cost of living, French books, and the Collection Armand Colin in particular, provide reliable technical information at a marvellously low price.

LUCIEN E. PICOLET.

METHODS OF MEASURING TEMPERATURE. By Ezer Griffiths, D.Sc., formerly Fellow of the University of Wales, Principal Assistant in the Physics Department of the National Physical Laboratory, with an introduction by Principal E. H. Griffiths, F.R.S. Second edition revised, xii–203 pages, with illustrations. London, Charles Griffin and Company, Limited; Philadelphia, J. B. Lippincott Company, 1925.

Temperature measurement is a recognized paramount factor in all investigations in physical science, and in present-day industrial processes as well, its importance has become supreme. Very high and very low temperatures are employed in these modern activities and rough approximations of their values will not suffice. We owe to the celebrated steel-maker, Hadfield, an early striking demonstration of this fact. He showed that carbon-steel, when quenched at a temperature below the recalescence point, failed to harden, but when quenched at a temperature only 15° C. above that point, it became totally hard.

The present volume describes and analyzes the details of the methods and apparatus for making temperature measurements which meet these modern requirements. The first chapter contains a general account of the determination of the fundamental scale of temperature. The calibration of mercurial, gas and resistance thermometers covering a range, in total, from –200° C. to 1500° C. are outlined along with references to the determination of the boiling-point of sulphur. The technique of calibration of mercurial thermometers occupies the second chapter. In the third chapter are described and critically examined successful forms of resistance thermometers with approved forms of resistance bridges. Specifications for standardization by the sulphur boiling-point with an example of their application to actual observations are included. The fourth chapter deals at equal length with thermocouple thermometers and their auxiliaries.

The succeeding chapters, which occupy slightly more than half the volume, relate to the laws of radiation and to radiation pyrometers. The "fourth power" law is given a summarized review. Accounts of the Féry, Foster and Thwing radiation pyrometers follow. Succeeding chapters are devoted to radiation from oxide and metallic surfaces, the distribution of energy in the spectrum of a "full radiator" and the principles of optical pyrometry, the distribution of energy in the heat-emission spectrum of the metals and, finally, the determination of high-temperature melting-points. An appendix gives specifications of sulphur boiling-point apparatus. The copious lists of bibliographical references at the end of each chapter is a highly commendable feature which is fortunately coming into increasing vogue in technical literature.

Although the work contains a considerable volume of analytical discussion, its strongest appeal is to the experimental investigator. Its presentation of a collection of the most modern, authoritative and generally accepted methods of measuring temperature constitutes a valuable document. The make-up of the book is of the usual style and high quality of the Griffin-Lippincott productions.

LUCIEN E. PICOLET.

NATIONAL ADVISORY COMMITTEE FOR AERONAUTICS. Report No. 222, Spray Penetration with a Simple Fuel Injection Nozzle. By Harold E. Miller and Edward G. Beardsley. 8 pages, illustrations, quarto. Washington, Government Printing Office, 1926. Price, five cents.

The tests covered by this report form a part of a general investigation of the application of fuel injection principles to aircraft engine service. The purpose of these tests was to obtain specific information on the rate of penetration of the spray from a simple injection nozzle, having a single orifice with a diameter of .015 inch when injecting into compressed gases.

The fuel was sprayed into a chamber fitted with glass walls and filled with nitrogen at various pressures. Special high-speed photographic apparatus, capable of taking a continuous series of fifteen photographs at a rate of 4000 per second, was used to record the development of single sprays. The effects of fuel pressures from 2000 to 8000 pounds per square inch and chamber pressures from atmospheric to 300 pounds per square inch on the rate of penetration and the development were studied.

The results have shown that the effects of both chamber and fuel pressures on penetration are so marked that the study of sprays by means of high-speed photography or its equivalent is necessary if the effects are to be appreciated sufficiently to enable rational analysis. It was found for these tests that the negative acceleration of the spray tip is approximately proportional to the 1.5 power of the instantaneous velocity of the spray tip.

Report No. 226, Characteristics of a Boat Type Seaplane during Take-off. By J. W. Crowley, Jr., and K. M. Ronan. 11 pages, illustrations, quarto. Washington, Government Printing Office, 1926. Price, five cents.

This report, on the planing and get-away characteristics of the F-5-L, gives the results of the second of a series of take-off tests on three different seaplanes conducted by the National Advisory Committee for Aeronautics, Navy Department. The single-float seaplane was the first tested (Reference 1) and the twin-float seaplane is to be the third.

The characteristics of the boat type were found to be similar to the single float, the main difference being the increased sluggishness and the relatively larger planing resistance of the larger seaplane. At a water speed of 15 miles per hour the seaplane trims aft to about 12° and remains in this angular position while plowing. At 22.5 miles per hour the planing stage is started and the planing angle is immediately lowered to about 10°. As the velocity increases the longitudinal control becomes more effective, but overcontrol will produce instability. At the get-away the range of angle of attack is 19° to 11° with velocities from the stalling speed through about 25 per cent. of the speed range.

Report No. 231, Investigation of Turbulence in Wind-tunnels by a Study of the Flow about Cylinders. By H. L. Dryden and R. H. Heald. 17 pages, illustrations, quarto. Washington, Government Printing Office, 1926. Price, ten cents.

With the assistance and coöperation of the National Advisory Committee for Aeronautics the Bureau of Standards has been engaged for the past year in an investigation of turbulence in wind-tunnels, especially insofar as turbulence affects the results of measurements in different wind-tunnels.

Two methods of making such studies are described in this report together with the results of the use in the 54-inch wind-tunnel of the Bureau of Standards. The first method consists of measuring the drag of circular cylinders; the second of measuring the static pressure at some fixed point. Both methods show that the flow is not entirely free from irregularities.

QUANTUM PRINCIPLES AND LINE SPECTRA. By J. H. Van Vleck, Assistant Professor of Physics, University of Minnesota. Bulletin No. 54, March, 1926, Vol. 10, part 4, of the National Research Council. 316 pages, 8vo. Washington, D. C., National Research Council, 1926. Price, $3.

This bulletin is a survey of the fundamental postulates and recent developments in the quantum theory of line spectra, with references to the literature and to the work of over 250 different authors. Special emphasis is placed on Bohr's correspondence principle as a focal point for much of the discussion. Among the subjects treated are models of the helium atom and hydrogen molecule, the polarization of resonance radiation, virtual oscillators, quantum theories of dispersion, and corpuscular light-quanta. As far as possible the mathematical development is reserved for a chapter on mathematical technique in the quantum theory, in which pertubation theory methods, etc., are given. A particular effort is made to correlate and contrast the rather confusing different systems of quantization now in use. Duplication of Sommerfeld's "Atombau and Spektral-linien" is avoided, as far as practicable.

PUBLICATIONS RECEIVED.

Metallurgy and Its Influence on Modern Progress, with a Survey of Education and Research, by Sir Robert A. Hadfield, Bt. 388 pages, plates, portraits, 8vo. New York, D. Van Nostrand Company, 1926. Price, $7.50.

Introduction to Theoretical Physics. Vol. II, by Arthur Haas, Ph.D., Professor of Physics in the University of Vienna. Translated from the third and fourth editions by T. Verschoyle, M.C., B.Sc. 414 pages, illustrations, 8vo. New York, D. Van Nostrand Company, 1926. Price, $6.

La Théorie de la Relativité, par M. von Laue, Professeur de Physique théorique à l'Université de Berlin. Traduction faite d'après la quatrième édition allemande revue et augmentée par l'auteur par Gustave Létang, Ingénieur ancien élève de l'École Polytechnique. Tome II, La relativité générale et la théorie de la gravitation d'Einstein. 318 pages, illustrations, 8vo. Paris, Gauthier-Villars et Cie., 1926. Price, 65 francs.

A Numerical Drill Book on Physics, by William Lord Taylor. 95 pages, illustrations, 12mo. Boston, Ginn and Company, 1926. Price, $1.

Les Moteurs à Explosion, par Edmond Marcotte, Ingénieur Conseil, Professeur à l'École spéciale des Travaux Publics. 216 pages, illustrations, 16mo. Paris, Librairie Armand Colin, 1926. Price, in paper, 7 francs.

The Principles of Physical Optics. An historical and philosophical treatment by Ernst Mach. Translated by John S. Anderson and A. F. A. Young. 324 pages, illustrations, plates, 8vo. New York, E. P. Dutton and Company. 1926. Price, $6.

Ontario Department of Mines. Thirty-third Annual Report, part 1, 1924.

Statistical review of Ontario's mineral industry in 1923 by W. R. Rogers. 63 pages, 8vo. Toronto, King's Printer, 1925.

U. S. Department of Commerce. Revised Simplified Practice Recommendation No. 16, Lumber. Issued by the Bureau of Standards, revised drafts, effective from July 1, 1925. 87 pages, illustrations, 8vo. Washington, Government Printing Office, 1926. Price, fifteen cents.

Bell Telephone Laboratories. Reprints of papers: Joseph Henry, by Bancroft Gherardi and Robert W. King. 10 pages, illustrations, 8vo. Development and Research in the Bell System, by E. B. Craft. 15 pages, 8vo. New York, Bell Telephone Laboratories, Inc., 1925–1926.

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Measurement of the Refraction of X-rays in a Prism by Means of the Double X-ray Spectrometer. BERGEN DAVIS and C. M. SLACK. (*Phys. Rev.*, Jan., 1926.)—For a long time vain efforts were made to discover refraction in the case of X-rays and now within two years the effect has been measured by several investigators. The method here described depends upon the reflection of a beam of monochromatic X-rays first from one, then from a second crystal surface, the two surfaces having been in contact before the cleaving of the piece of Iceland spar within which they were. Observations are made with an ionization chamber when the second reflecting surface is rocked through a small angle. Then a prism of aluminum is placed in the path of the beam after its first reflection and the observations with rocking are repeated. From the difference between the results obtained with and without the prism, it is possible to calculate to an accuracy of 5 per cent. the index of refraction of aluminum for the X-rays used. For those of wave-length .7078 Å.U. this was 1.68×10^{-6} less than 1 and for 1.537 Å.U., 8.4×10^{-6} less. These values agree well with those derived from Lorentz's dispersion formula. G. F. S.

Freezing-points. H. W. FOOTE and GENEVA LEOPOLD. (*Am. Jour. Sci.*, Jan., 1926.)—The freezing-point of a liquid varies slightly according to the amount of air dissolved in it. There is about $1/30^{\circ}$ difference between the freezing-points of air-free and air-saturated benzene at the pressure of one atmosphere. It is, however, reassuring to read that "there are but few one-component freezing-points known at present with such accuracy that the error due to uncertain saturation with air would affect the results." The authors furnish a list of all compounds whose freezing-points are determined to an accuracy of .05°. There are no more than twelve, of which water is one. The freezing-point of water is lowered by the presence of air in solution to saturation to the extent of .002°.

Since exact determinations of the freezing-point have usually been made with the liquid largely or entirely saturated with air and

since, further, saturation with this gas takes place rapidly, the authors "prefer to regard a standard freezing-point as the temperature at which solid and liquid are in equilibrium with dry air at a pressure of one atmosphere rather than the temperature at which the two phases are in equilibrium at a pressure of one atmosphere, which is the usual definition, and which makes the standard freezing-point of water uncertain to the extent of .0023°." G. F. S.

Tests of Relativity Theory. A. S. EVE. (*Nature*, April 10, 1926.)—When Professor Eve attended the meetings of Convocation Week last December in Kansas City, he heard so much relating to the theory of relativity that he gave a lecture on the subject to the Physical Society of McGill University after his return to Montreal.

There are five experimental tests for the theory of relativity:

(1) The shift of the perihelion of the planet Mercury amounted to 41 seconds of arc per century and was without explanation until Einstein showed that it should be 43 seconds according to his theory. Now fresh calculations of the observed value put it at angles varying from 29 seconds to 38 seconds and thus demand renewed explanation.

(2) The deviation of rays of light passing close to the sun. This effect, which can be measured at the time of total eclipses of the sun, is well established "but there are many astronomers and physicists who are well aware that this verified effect may be assigned to a number of causes, particularly to refraction."

(3) The shift toward the red in the wave-length of light emitted by the sun. This increase of wave-length is amply confirmed by St. John and Evershed. In the case of Sirius, and its companion, the White Dwarf, the shift is more than thirty times as great as in the sun. Apropos of this it is somewhat startling to read of a star that has a density 50,000 times that of water.

While Einstein himself holds without qualification to this shift as a necessary consequence of his theory, Silberstein, on the contrary, in what Professor Eve calls "his exquisite treatise on 'The Theory of Relativity,'" disagrees with him on this point.

(4) Relative motion of the earth through space. The theory of relativity arose as an explanation of the failure of the Michelson-Morley experiment to detect any motion of the earth through the ether. Now comes Dayton C. Miller, who knows more about the technique of this experiment than anyone else and announces as the outcome of many observations made on Mount Wilson that there at least the earth is sweeping through space with a speed of 10 km. per sec. Little wonder that Professor Eve observes, "Mixing metaphors, we see the very experiment which first started the ball a-rolling, turning round to bite the hand which fed it." In view of Professor Miller's inability to detect any relative motion of the earth to the space in a basement in the city of Cleveland and other American physicists are reported to be planning to repeat the experiment at different elevations above the sea. "It appears true that if Miller's

results are confirmed at different places, then all our theories go into the melting-pot, relativity and all, and a patient resurvey must commence to unravel afresh order from chaos. The only grave mistake we can make at present is to treat Miller's results otherwise than seriously."

(5) The Chicago rotation experiment, in which two rays of light were sent around the same path but in opposite directions. The results are equally in accord with the relativity theory and with the theory of a stagnant ether, but they are in opposition to Miller's suggestion that the earth drags 95 per cent. of the ether with it while 5 per cent. of it slips by. Be it noted that the fact of Miller's observations are quite apart from his suggested explanation of them.

Thus for the theory of relativity are the motion of the perihelion of Mercury, deviation of rays near the Sun and shift of spectral lines. Against it stand Miller's results, while the Chicago rotation experiment holds a neutral position.

C. L. R. E. Menges, of Scheveningen, Holland (*Science*, April 23, 1926), maintains that the Fizeau experiment furnishes the real test for the theory of relativity. This has recently been repeated by P. Zeeman in Amsterdam. He claims that "Zeeman's result is the decisive proof of Einstein's theory."

From all above set forth it seems that an interesting period for the relativity theory lies ahead. G. F. S.

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